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27 March, 1952

Mr V. A. M. Robertson, C.B.E., M.C., Past-President I.C.E., in the Chair

The following Paper was presented for discussion and on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Railway Paper No. 45

" Recent Developments in Railway Curve Design "

by

Joseph Charles Loach, M.Sc.(Eng.), A.M.I.C.E.,
and Martin George Maycock, B.Sc.(Eng.), A.M.I.C.E.

SYNOPSIS

When any vehicle changes its forward direction of motion a sideways force known as "centrifugal force," comes into play; its magnitude is affected by the forward speed of the vehicle and the radius of the curve. In railway curve design it is essential that both these items should be considered, together with (1) cant of the track; (2) rate of gain of the cant; (3) cant gradient; (4) cant deficiency; and (5) rate of gain of cant deficiency.

Until the present time, there has been no uniformity in the practices of civil engineers with regard to the limiting values used for most of the above features. The limits may be defined by safety considerations or by the desire to keep the sensations which they give to passengers within reasonable standards of travel comfort.

In 1949, the Track Committee of the Railway Executive of British Railways was given the remit of defining the limiting values which should be used for each of the five items mentioned. Whilst the first of them was readily agreed upon, the others were not so easily settled and it soon became apparent that some tests should be made.

The first part of this Paper describes all the tests which were eventually conducted, and gives the results obtained; it also gives the limiting values which were finally recommended by the Track Committee for the other four items mentioned above.

In the second part of the Paper, rules are developed for the equilibrium speed and the maximum permissible speed on curves in relation to the radius of curve, cant, and length of transition. They incorporate the limiting values decided on as a result of the tests.

The rules are applied not only to simple curves but also to compound and reverse curves, and to the more complicated case of curves through junctions and crossovers.

Graphs are presented which greatly facilitate the application of the rules, and numerous examples of their use are given.

Part I.—Experiments to Determine Data for the Design of Curves

by

J. C. Loach, M.Sc.(Eng.), A.M.I.C.E.

INTRODUCTION

FOR many years the design of curves and particularly that of the entrances and ends of curves was not well defined, most railway engineers having held different views on the subject. In the early days of railways it was recognized that cant was desirable on curves but the factors on which it depended were not generally understood; much more recently, opinions on the amount of cant which there should be on a curve have differed, and cases have occurred of the cant on a curve being materially altered to comply with the ideas of one engineer and then being put back again by his successor.

Even more diverse were the views held on transitions when they were used at all. As long ago as 1868 [1], a curve of cubic form was advocated. At the turn of the century, Mr James Glover [2] found that the perfect transition, a curve in which the radius at any point was inversely proportional to the distance of that point from the origin, was a spiral defined by the equation $\lambda = m\sqrt{\phi}$. However, Glover also mentioned that when ϕ was not more than 10 degrees, there was no appreciable difference between the spiral and the cubic parabola. A decade later, Mr W. H. Shortt [3] used the spiral in his methods of improving the alignment of curves; but in the course of time it was realized that most transitions could be designed without ϕ exceeding 10 degrees, in which case they approximate to cubic parabolas. During the following 30 years designed transitions became general on the main lines in Britain, partly because the benefit of them was becoming more appreciated and partly through the adoption of methods quicker and simpler than those involving the use of a theodolite for setting them out.

In the 1930's, the introduction of some high-speed trains in Britain involved a wider range of speeds. Mr Taylor Thompson [4] explained that curves had been adjusted to suit best the faster trains, but that the fastest trains were running at speeds involving "cant deficiency" for which he suggested a limiting permissible value dictated by the comfort of the passenger. He also suggested that it seemed most rational for the length of the transition to be a function of the speed. However, the limiting values for various items involved in curve design have remained matters of opinion, and the Author submits that the tests described in Part I of the Paper have helped considerably to clarify the matter and have enabled suitable values to be established.

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[1] The references are given on p. 523.

The Initiation of a Unified System

Very soon after the nationalization of British Railways the desirability of having one system only for curve design was realized, and the necessity immediately arose for establishing limits of various features. Mr J. C. L. Train, Member of the Railway Executive for Civil Engineering and Chairman of the Civil Engineers' Committee, asked the Chairman of the Track Committee for that Committee to recommend a code of practice.

Firstly, the practices of the four former main-line companies were summarized; at the same time, information was sought on the rules and limits to which railway engineers worked in other parts of the world. Simultaneously, all Regional Civil Engineers were consulted for their views on some suggested limits which were, more or less, the limits that had been in use for a few years on the former London and North Eastern Railway. The information was reviewed by the Track Committee who felt that it was desirable to establish:

the maximum permissible cant;
the maximum rate (with respect to time) of gain and loss of cant;
the maximum permissible cant deficiency;
the maximum rate of gain and loss of cant deficiency; and
the steepest permissible cant gradient,

which should be used in design.

First Discussions

The two main factors in the problem were: (1) safety and (2) comfort.

Maximum Permissible Cant

When discussing cant, one or two cases were mentioned of four-wheeled wagons which had been standing on curves with 6 inches of cant* and of the outside leading-wheel flanges climbing the rail when they started to move. A case was also cited of a van in a similar position which had been overturned by a gale blowing sideways on to it. Both cases were regarded as exceptional, but they indicated that it would be unwise to cant a track more than 6 inches. It was agreed that satisfactory maintenance of track becomes more difficult as the cant increases above 4 or 4½ inches, and since 6 inches was already regarded as the maximum permissible amount on most railways there was little difficulty in agreeing on that figure.

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* In this Paper, all numerical values relate to standard track of 4-foot-8½-inch gauge.

Amount of Cant to be Provided

It was also agreed that, wherever possible, the cant should give equilibrium conditions at the speed selected as the basis for calculation. The determination of the speed for any particular case is discussed in detail in Part II.

Other Features

Other points to be settled proved to be far more difficult: practices on the four main-line companies varied considerably, as did also the views of the Regional Civil Engineers with regard to the limits suggested; the information from railways abroad showed no uniformity; in discussion it soon became evident that most members of the Track Committee had different views on the degrees of discomfort brought about by various cant deficiencies and various rates of gain of cant and of cant deficiencies; nor could anyone define the upper limits which it would be reasonable to expect a passenger to tolerate. It was realized, however, that unbalanced centrifugal force would be uncomfortable to the passenger at values considerably lower than those at which the train would be in danger of overturning, and thus the criteria should be based on sensations experienced by the passenger. With the object of establishing those criteria it was decided to conduct some tests.

PRELIMINARY TESTS

A kind offer from the engineers of the London Transport Executive to make some starting and stopping tests on an electric train and on a trolley-bus, was accepted. The tests were made on the 8th April, 1949; members of the Track Committee sat on seats arranged longitudinally in the vehicles and, in each case, the driver endeavoured to accelerate or decelerate at a constant rate. These tests, regarded as a preliminary to some later tests, proved to be most helpful; the results obtained in a two-car electric train were as given in Table I.

TABLE 1.—RESULTS OF PRELIMINARY TESTS WITH ELECTRIC TRAIN

Miles per hour per second	Sensations
1·0	Just noticeable
1·3–1·5	Noticeable
2·0	Pronounced but not objectionable
2·3–2·5	Quite pronounced but not at all uncomfortable
2·8	Strong and slightly uncomfortable
3·4	Rather uncomfortable

One or two tests were also made while members of the Committee were standing; the sensations experienced showed that 2·4 miles per hour per second was about the limit which could be attained without giving discomfort and that 2·7 miles per hour per second was somewhat uncomfortable.

During the tests the rate of increase of the deceleration was observed to have an influence on the sensations, the “ passengers ” being able to withstand more easily a deceleration when there had been a more gradual increase of its rate.

Tests on the trolley-bus were made on straight level road, but because of the lower initial speeds, it was not easy to maintain a constant deceleration for long. Several tests were made with decelerations ranging from 4 to 15·5 feet per second per second. The general opinion of the Committee, both when members were sitting and when they were standing, was as shown in Table 2.

TABLE 2.—RESULTS OF PRELIMINARY TESTS WITH TROLLEY-BUS

Feet per second per second	Sensations
4	Pronounced and barely comfortable, especially when there had been a rapid increase in deceleration to that value
5	Somewhat uncomfortable
6	Uncomfortable

Summarizing, the Committee felt that the division between comfort and discomfort on the train was at 2·4 miles per hour per second (equivalent to 3·5 feet per second per second and to 6·5 inches cant deficiency); on the trolley-bus this division was at 4 feet per second per second (equivalent to 7·3 inches cant deficiency).

The Track Committee discussed the results and whilst appreciating that conditions were somewhat different from those experienced in normal travel, they felt that the results had significance and that the subject was worth pursuing, particularly with the additional aim of establishing limiting values for the rates of gain and loss of cant and of cant deficiency. They then asked the Author to organize some trials which would involve travelling round railway curves at speeds giving up to 6 or 7 inches cant deficiency.

PRINCIPAL TESTS

Details of Curves

On the suggestion of Mr J. Briggs, then Civil Engineer of the London Midland Region, the length of track between Bettws-y-Coed and Blaenau Festiniog in North Wales was inspected and was soon realized to be very suitable for the tests. Between mile-posts 15 and 24½ there are fifty-two curves of radii varying between 9 and 80 chains, but mostly 10 to 13 chains. The Festiniog branch is single line and on the proposed test portion there is a general speed restriction of 30 miles per hour. Because of the rather severe gradients, it was much easier to conduct the tests when the train was running downhill around the curves, that is, from Blaenau Festiniog to Bettws-y-Coed.

The curves were numbered consecutively and each was divided into three portions, the two transitions with the circular part between them. The points of division were located by boards on stakes driven into the ground at the line-side.

Curves 1 to 14 were set out very carefully with line and cross-level, in accordance with methods normally used on the London Midland Region. Taking as a basis an equilibrium speed of 30 miles per hour, a chord-length of 48 feet 4 inches was used so that the versine on it and the cant of the track would be equal; on the transitions the increments of both were made proportional to the distance along the transition (that is, "straight-line versine and cant gradient" diagrams plotted on a distance base), adjustments of versines being made at the ends.

Curves 15 to 52, on the other hand, although not specially set out, were rather better than one would normally expect to find on a branch line; a few had straight-line approaches which gained some of the cant, the remainder being gained on the circular curves themselves, a practice in vogue with the former London and North Western Railway Company when they constructed the line in the nineteenth century, whilst others had the ends of the curves eased off into the straight approaches, thereby giving a form of transition in alignment.

The circular parts of all of curves 1 to 14 and of many of the others were very regular; by applying the formula devised by Mr E. T. Hutt [5] to define the regularity of alignment of circular curves, it was found that those which had been carefully set out gave values averaging 93 per cent, whilst of curves 15 to 52 which were measured, nearly all gave values between 84 and 95 per cent, several giving more than 90 per cent.

Details of Train and Test Runs

The tests were conducted on the 11th October, 1949; the test-train consisted of a 2–6–2 tank locomotive, one unit of the Mobile Locomotive Testing Plant [6], a vestibule coach in which sat the twelve members of the Track Committee and one other observer, and lastly the Saloon of the District Engineer.

Three test runs were made, speeds being:

(1) 40 miles per hour,
(2) 30 ,, ,, ,, (the equilibrium speed),
(3) 45 ,, ,, ,, * over curves Nos 1 to 14, and 42 miles per hour over curves Nos 15 to 52.

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* The full speed of 45 miles per hour was not attained until the train reached curve No. 5.

On each occasion a stop was made at Dolwyddelen, where there is a passing loop, between curves 23 and 24.

On each of the three runs, speed was maintained constant by the Mobile Testing Unit and, as the journey progressed, the numbers and portions of the curves were announced from a loud speaker in the vestibule coach. Each of the thirteen observers sitting in the coach recorded his sensation of comfort on the transitions and on the circular part of each curve.

Code of Sensations

To assist the observers in writing down their sensations quickly and to obtain uniformity in the expression of their sensations within an approved range, a code of sensation numbers was adopted as shown in Table 3.

TABLE 3.—SENSATION NUMBERS ADOPTED

Number	Sensation
0	Nothing noticed
1	Just noticeable
2	Noticeable
3	Pronounced
4	Very pronounced but not at all uncomfortable
5	Strong and slightly uncomfortable
6	Rather uncomfortable

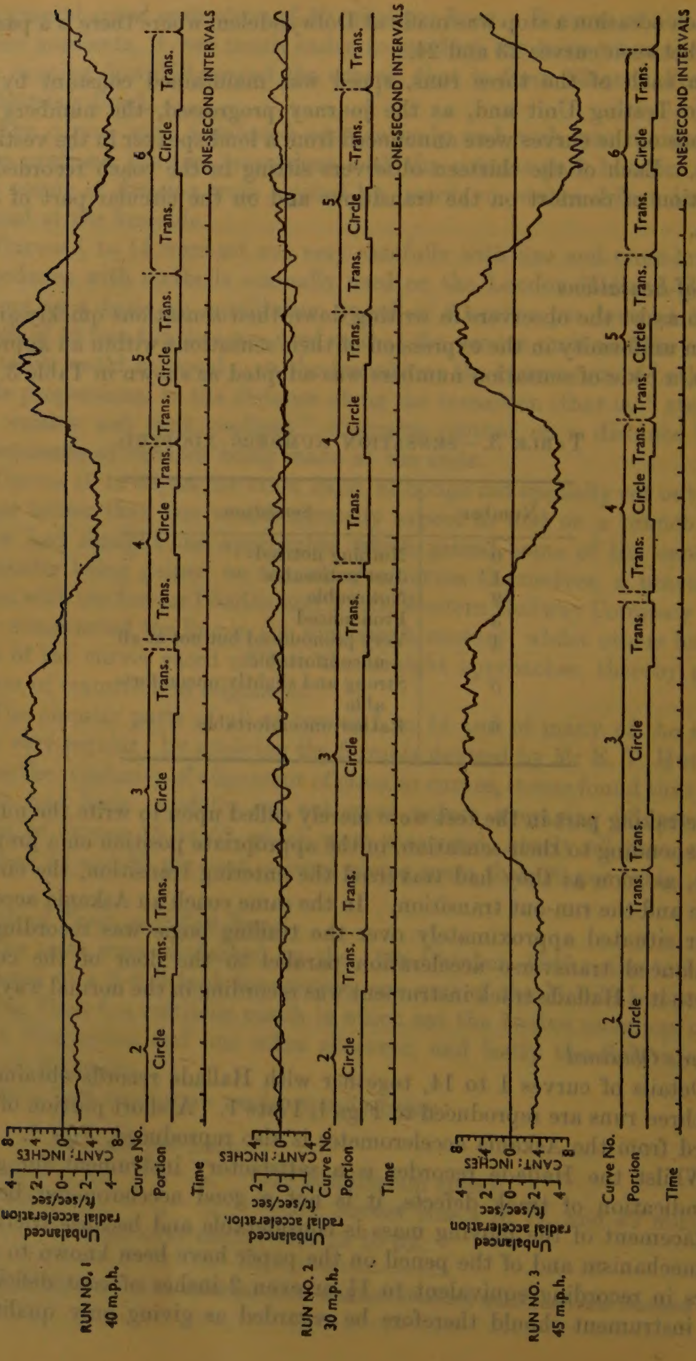
Those taking part in the test were merely called upon to write the number corresponding to their sensation, in the appropriate position on a prepared sheet, as soon as they had traversed the entering transition, the circular curve and the run-out transition. In the same coach an Askania accelerometer situated approximately over the trailing bogie was recording the unbalanced transverse acceleration parallel to the floor of the coach; near to it a Hallade track instrument was recording in the normal way.

Records Obtained

Details of curves 1 to 14, together with Hallade records obtained on the three runs are reproduced in Figs 1, Plate 1. A short portion of each record from the Askania accelerometer is also reproduced, *Figs 2*.

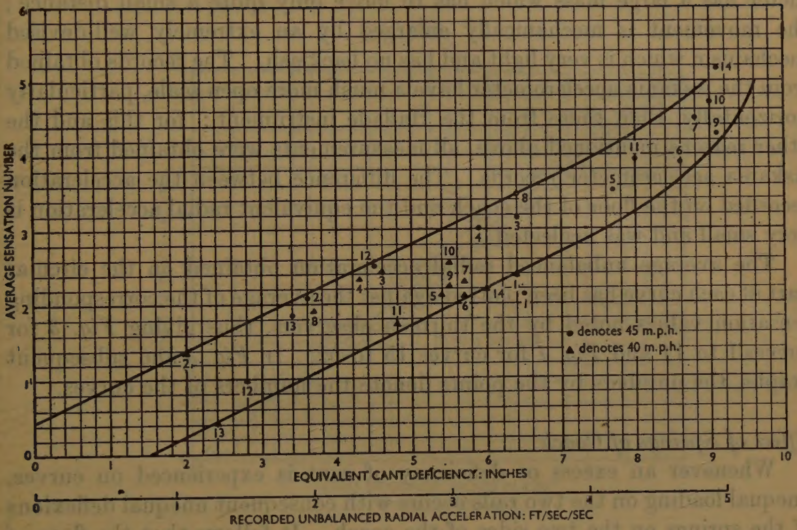
Whilst the Hallade recorder is a satisfactory instrument for giving an indication of track defects, it is not a good accelerometer because displacement of the moving mass is considerable and because friction of the mechanism and of the pencil on the paper have been known to cause errors in recording equivalent to $1\frac{1}{2}$ or even 2 inches of cant deficiency. The instrument should therefore be regarded as giving only qualitative

Figs 2



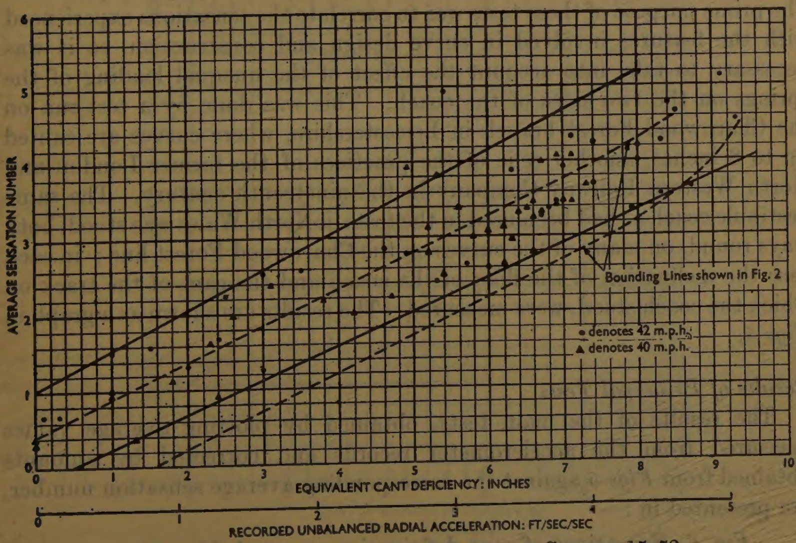
REPRODUCTION FROM ASKANIA ACCELEROMETER

Fig. 3



Sensations Experienced on Circular Curves 1–14

Fig. 4



Sensations Experienced on Circular Curves 15–52 and not quantitative results. On the other hand, the Askania accelerometer has a large mass which has to move only quite a small distance; the movement is mechanically enlarged by an extremely well-devised mechanism which is very light and has no backlash. The records obtained from the Askania accelerometer have a much more open scale, particularly horizontally, than those from the Hallade instrument: for this and the other reasons mentioned above, all measurements were obtained from the Askania accelerometer records. The difference between the acceleration recorded on the floor of the coach and the equivalent radial acceleration is very small and was neglected.

The average unbalanced radial acceleration obtained on the circular part of each curve has been plotted against the average of the corresponding sensation values noted by the thirteen observers, thus giving *Fig. 3* for curves 1 to 14 and *Fig. 4* for curves 15 to 52. In *Fig. 3* and subsequent graphs, the numbers by the points denote the numbers of the curves.

Effect of Springs of Coach

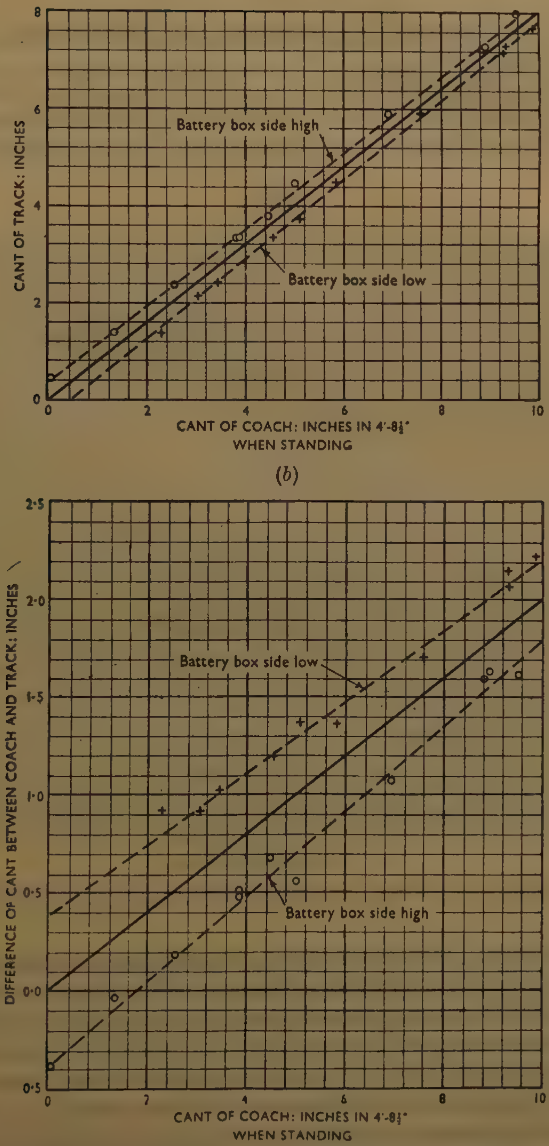
Whenever an excess or deficiency of cant is experienced on curves, unequal loading on the two rails occurs with consequent unequal deflexions of the springs on the two sides of the coach. It follows that the floor of the coach is inclined more or less steeply than the plane of the track under it; any accelerometer on the floor is activated by the centrifugal force that is not balanced by the component of gravity due to the inclination of the floor, and so includes the effect of the springs in the record it produces. The prime purpose of these tests was to correlate the sensations experienced with the features involved in curve design and construction, so it was necessary to take into account the effect of the unequal loading of the springs on the two sides of the coach. This was done by a test-run on the Charnwood Forest branch in Leicestershire, where curves are canted up to 8 inches, another relic of the practices of the former London and North Western Railway Company in the nineteenth century. The same vestibule coach as had been used in the tests in North Wales was stood, both ways round, on many of the curves on the Charnwood Forest line; in each case the inclination of the floor of the coach and the cant of the track on which the coach stood, were measured. The results are shown as a graph in *Figs 5*.

Results of Principal Tests

The results of the main tests, obtained by plotting average values measured from the accelerometer records and corrected by amounts obtained from *Figs 5* against the corresponding average sensation number, are presented in:—

Fig. 6. Sensations of cant deficiencies, curves 1–14.
Fig. 7. Sensations of cant deficiencies, curves 15–52.
Figs 8 (*a*) and (*b*). Sensations of gain of cant deficiencies.

Figs 5


(a)

(b)

Effect of Unequal Loading on Springs of Coach

Figs 9 (*a*) and (*b*). Sensations of loss of cant deficiencies.

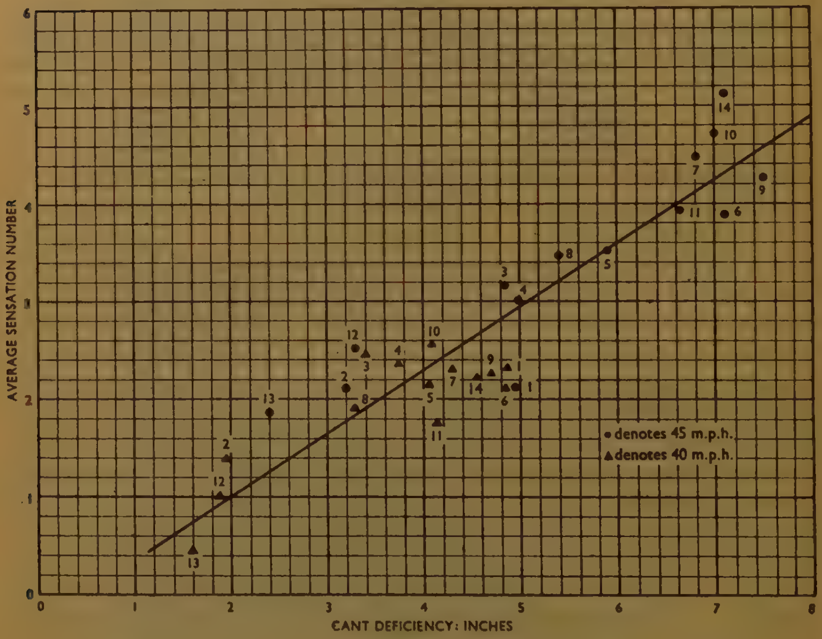
Figs 10 (*a*) and (*b*). Sensations of gain and loss of cant at equilibrium speed.

Examination of Results, and Deductions

Cant Deficiencies

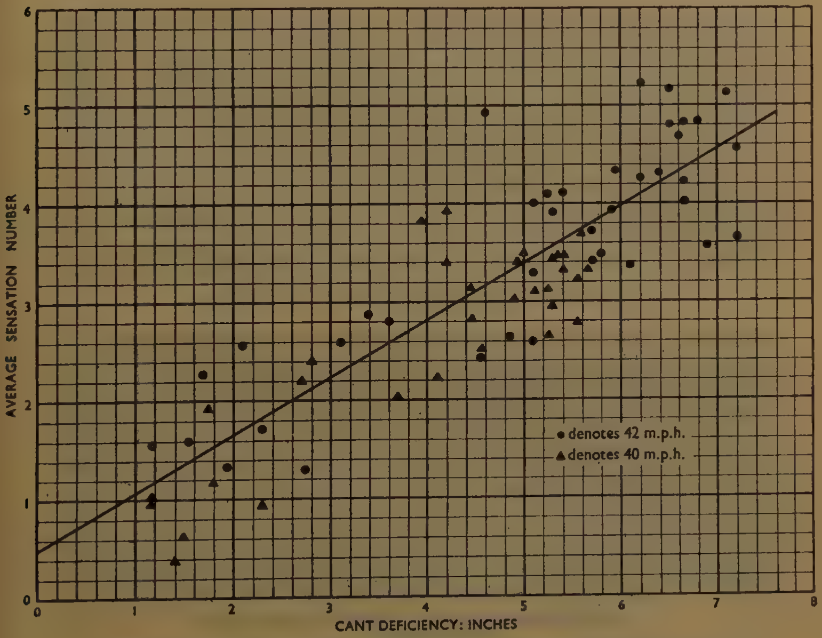
Fig. 6 shows that discomfort was not experienced until the deficiency of cant was as much as 7 inches and, in this respect, the results of the trials in North Wales confirm very satisfactorily those obtained from the preliminary tests. It was recognized, however, that standing passengers would experience discomfort at deficiences of less than 7 inches and that passengers who were trying to walk along corridors or through luggage vans would be inconvenienced at even smaller deficiencies. It was also recognized that the curves for which a greater permissible cant deficiency would be most desirable were the sharper ones ; in these cases a small increase in speed would increase appreciably the cant deficiency and the lateral thrusts, and result in increased maintenance and shorter life of track. At the same time, the sensations of discomfort would be increased perhaps even more than the cant deficiency. Consideration also had to be made of the

Fig. 6



Sensations of Cant Deficiencies on Circular Curves 1–14 ability of rail fastenings to withstand indefinitely the lateral thrusts which would be imposed on them when the fastest trains were running regularly at speeds associated with appreciable cant deficiencies, and of the problem of maintaining good alignment. In the end, these two items influenced the Committee perhaps more than any other because, apart from a few abnormal cases where exceptional circumstances justify the frequent renewal of the track, reasonably long life of the track is essential for reasons of economy.

Fig. 7



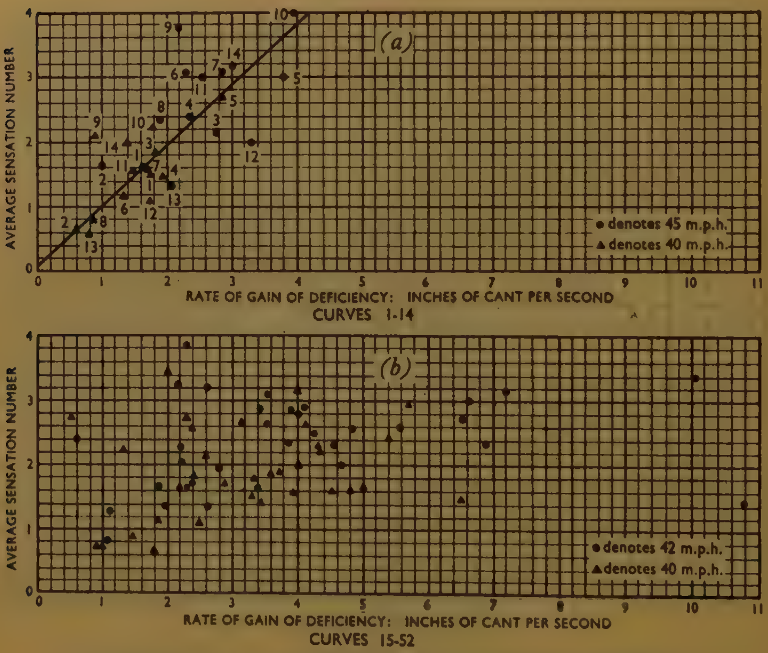
SENSATIONS OF CANT DEFICIENCIES ON CIRCULAR CURVE 15–52

The Track Committee therefore decided that 3½ inches cant deficiency, corresponding to sensation No. 2 (" noticeable "), was the maximum which should be tolerated in the design of curves in the more important and better maintained lines, that is, lines falling within the Classes defined as " A " and " B " (see Appendix to Part II, p. 541). From *Fig. 7* the corresponding degree of discomfort on curves not so well set out was given by 2½ inches cant deficiency, and the Committee adopted this figure for lines classed as " C " and " D " and for all turnouts. This lower scale was considered to be appropriate for Classes " C " and " D " lines because lighter or sometimes " second-hand " material from the more important lines is used in their construction, and because standards of maintenance are not normally so high as on Classes " A " and " B " lines.

In turnouts, transitions cannot often be incorporated so that the entrance directly on to a circular curve imposes a sudden lateral thrust on the passenger, the vehicle, and the track; the Committee therefore felt that it was not desirable to exceed 2½ inches cant deficiency in these cases.

In deciding on these figures, the Committee made allowance for drivers' errors in estimating their speeds where restrictions exist, the allowance having been deduced from a large number of observations made by the former London and North Eastern Railway Company, and in this connexion the Committee also decided to make the permissible cant deficiency a function of the cant. In the case of the more important lines the maximum permitted deficiency decreases by a straight-line law from 3½ inches at zero cant to 2 inches at 6 inches (maximum) cant. The corresponding figures for the less important lines and for curves in turnouts are 2½ inches at zero cant to 1 inch at 6 inches of cant. The reduction of permissible deficiency in design as the cant increases was thought to be justified by the greater influence of track imperfections at the high speeds

Figs 8

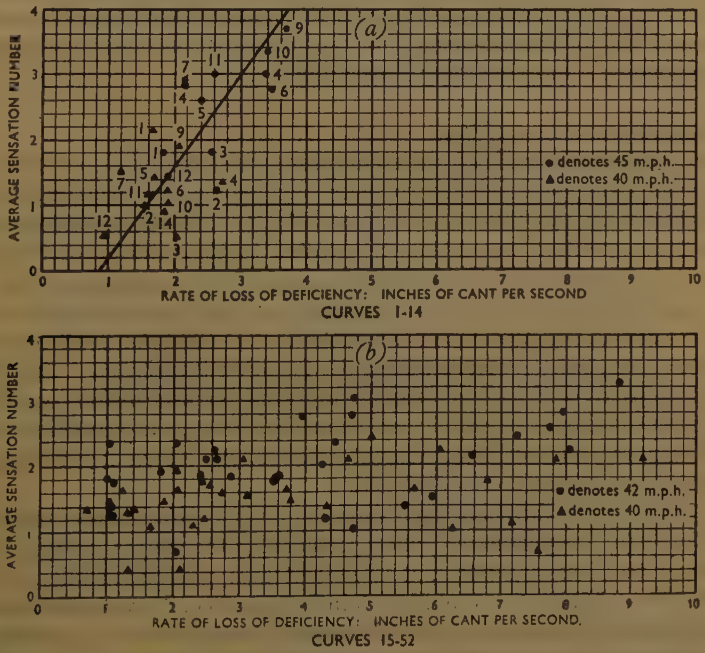


SENSATIONS OF CANT DEFICIENCY ON ENTERING TRANSITIONS associated with the higher cants. The relation is empirical but until more is known about the numerous factors involved, the formulation of any rule taking them into account would be a matter of guesswork; meanwhile, the relation between cant and the maximum permitted deficiency is simple, easy to use and one which observation and experience have shown to be justified.

Rate of Gain and Loss of Cant Deficiencies

It is recognized that while running over the transitions in test-runs 1 and 3, cant was being gained and lost at the same time as cant deficiency. Disregarding the effect of changes of cant, *Figs 8* (*a*) and *9* (*a*) indicate that when transition curves are set out by measurement, there is a relation between the rates of gain and loss of deficiency and the sensations experienced by the observers travelling round them. A loss of cant deficiency gives a sense of relief from discomfort to a passenger and, as might therefore be expected, sensation numbers recorded for the lower rates of loss of deficiency were somewhat less than those recorded for the same values of gain of cant deficiency. On the other hand, the higher rates of gain and loss of deficiency were equally unpleasant to the observers.

Figs 9



Sensations of Loss of Cant Deficiency on Run-out Transitions

Figs 8 (*b*) and *9* (*b*) show that transitions which have not been set out properly cannot be correlated. As mentioned earlier, part of the cant had already been gained on the straight approaches to many of the curves, an effect which the Author suggests is not quite the same to the passenger as that of sensing a gradually changing centrifugal force, notwithstanding the difference in direction of the unbalanced force. On each of a few transitions the gain or loss of deficiency was confined to one portion of it and was so rapid that the observers experienced a momentary lurch but only recorded an average sensation; this accounts for the scattered points towards the right-hand-side of the graphs. Having in mind much the same thought as when cant deficiency was discussed, the Track Committee decided to adopt for design purposes the maximum permissible rates of gain and loss of cant deficiency of 2¼ inches per second,* values again corresponding approximately to sensation No. 2.

Rate of Gain and Loss of Cant

In the test-run made at the equilibrium speed (30 miles per hour), the observers were not expecting to sense any unbalanced centrifugal force, particularly on the curves which had been set out by measurement, so that their attention was primarily devoted to the rotation of the coach about a longitudinal axis as it climbed or ran down the cant gradients.

Fig. 10 (*a*) shows that a relation exists between the rate of gain of cant and the sensations experienced when the transitions have been set out by measurement, but the results are erratic for other transitions. *Fig. 10* (*b*), referring to rate of loss of cant, gives rather erratic results on all the curves but it should be noted that, as in *Fig. 10* (*a*), the transitions which had been set out by measurement were mostly given lower sensation numbers than those not so designed.

On the whole, the points on the graph in *Fig. 10* (*b*) are lower than those in *Fig. 10* (*a*); nevertheless, when considering the points obtained from curves 1 to 14 on the latter, it is evident that passengers will only just notice rates of gain of cant of 2½ to 3 inches per second—values in excess of those which a curve designer would normally require to use. Incidentally, rates of gain of cant of this order prevailed during test-runs 1 and 3, and the results just presented show that to disregard the relatively slight effect on the observers of changes of cant was justified when the sensations caused by gain or loss of cant deficiencies were being considered.

For the great convenience of being able to make the fullest use of the graphs reproduced in Part II of this Paper, the Track Committee decided to limit the design value of the rate of gain of cant to 2¼ inches per second, this being the same as the limiting value of the rate of gain of cant deficiency for design purposes. It was emphasized, however, that a somewhat higher rate of gain of cant would not jeopardize the safe passage of trains.

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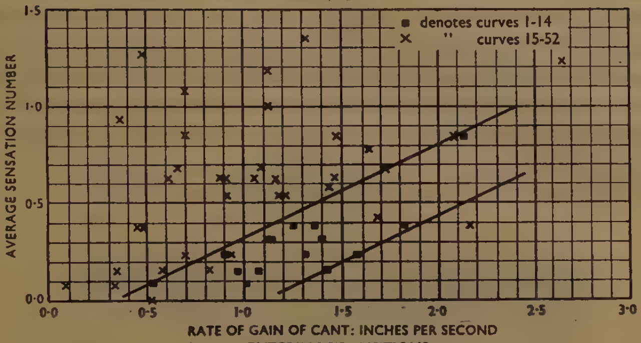
* Equivalent to a change of acceleration of 1·22 feet per second per second in a second.

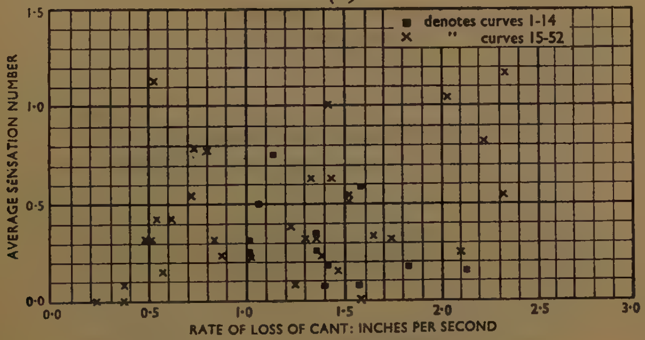
Figs 10

(a)



ENTERING TRANSITIONS

(b)



RUN-OUT TRANSITIONS

Sensations of Gain and Loss of Cant Transitions Traversed at Equilibrium Speeds (30 m.p.h.)

Cant Gradients

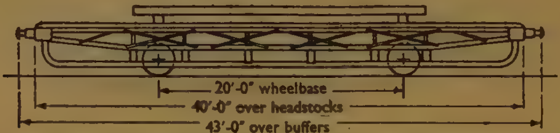
The term "rate of gain of cant" implies that the speed of a train is involved; but should the speed be low, gains of cant within the $2\frac{1}{4}$ inches per second allowed would result in steep cant gradients. The Track Committee therefore felt that a limit should be laid down for the steepness of cant gradients. When the matter was discussed, considerable diversity of views was again evident and members could do little more than express opinions.

The Author proposed some tests, which he was afterwards asked to carry out, to find the derailing tendencies of vehicles on cant gradients.

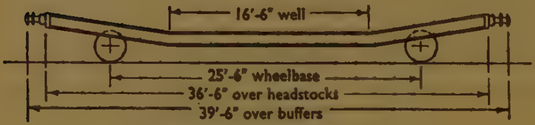
It can be shown theoretically that when a vehicle is traversing a length of curved track, there will be a tendency for the outer leading wheel of the vehicle to climb the rail, such a tendency depending on the ratio, for this particular wheel, of flange force P to wheel load W.

As mentioned earlier, when a vehicle moves round a curve at a speed other than the equilibrium speed, a transference of weight from the wheels on one side of the vehicle to those on the other takes place, owing to the unbalanced centrifugal force. A redistribution of weight will also occur when passing over transition curves, where a twisting effect will be introduced; this redistribution will be most pronounced when the vehicle is unloaded and, obviously, the longer the wheel-base the greater will be the twist applied. These two factors, therefore, will influence the load on any particular wheel, and consequently the ratio P/W.

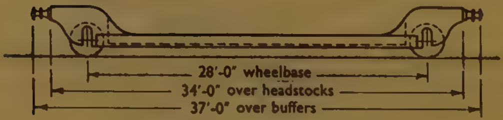
Figs 11



Height of C. of G. above rail: 2'-2"
CASEWAG ME No. 117289



Height of C. of G. above rail: 1'-9"
LOWMAC MR No. 904003



Height of C. of G. above rail: 1'-6"
WELTROL WY No. 41981

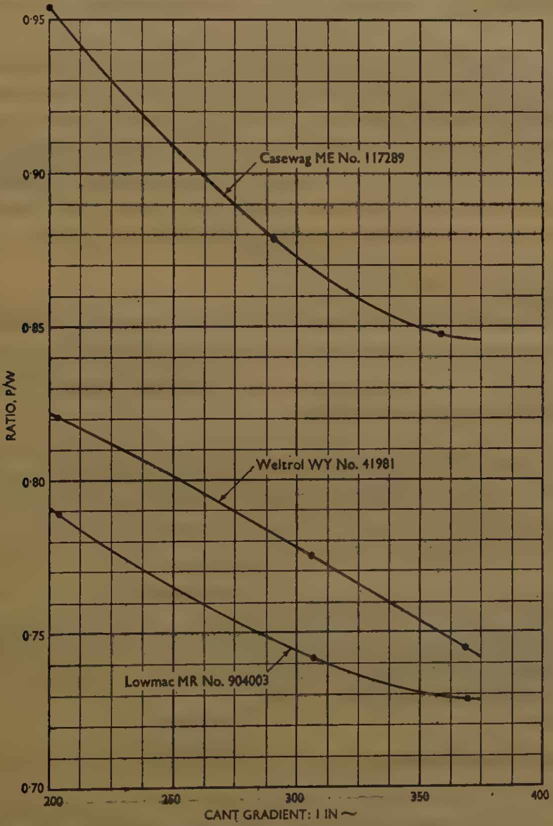
WAGONS USED IN TESTS TO FIND DERAILING TENDENCIES ON CANT GRADIENTS

It was thought that the vehicles whose weight distribution would be most affected would be those having the longest rigid wheelbases, or the stiffest frames and springs, or possibly a combination of these two features. A search through the wagon stock book of British Railways enabled the selection to be made of three types of wagons thought to be the most likely to be affected in the manner just mentioned. One wagon of each type was selected as representative of its kind; line diagrams of them are shown in *Figs 11*.

Tests were made on a weigh-bridge and consisted of measuring the individual wheel-loads of each wagon on artificial cant gradients, the gradients being obtained by running one wheel of the wagon being tested up a ramp on to packings of calculated thicknesses. For each wagon, unloaded, the wheel loads corresponding to cant gradients of approximately 1 in 200, 1 in 300, and 1 in 360 were thus obtained.

Afterwards, by removing the springs from the wagons and subjecting them to load-deflexion tests, it was ascertained that they were in average condition.

Fig. 12



Derailing Tendencies of Wagons on Cant Gradients
(Based on Theory of Überlacker: $\mu = 0{\cdot}25$, and calculated on $3\frac{1}{2}$ inches cant deficiency)

By considering the wheel-loads and the effects of cant in conjunction with the theories of Überlacker,[7] it was found that, for these wagons, the maximum value of P/W would occur when the cant deficiency was greatest. The transfer of weight from the inner to the outer wheels was therefore calculated for a $3\frac{1}{2}$-inch deficiency of cant. By using these values in conjunction with those obtained from the tests, the wheel-loads of each vehicle when travelling over the above-mentioned cant gradients with 3½-inch cant deficiency were determined and then used to calculate, by applying Überlacker's theory, the flange force *P* for each case.

Graphs of the values of the ratio *P*/*W* at the outer leading wheel and for the various cant gradients are shown in *Fig. 12*.

It will be observed from the graphs that the Casewag vehicle, though having a shorter wheelbase, has a greater tendency to derail than the other two types, presumably, because of the relatively stiffer framework of the wagon.

The Track Committee decided that the results, excluding those for the Casewag vehicle, justified the adoption of 1 in 300 as the value for the steepest permissible cant gradient. This figure would allow a reasonable margin of safety for any small discrepancy that might exist in the track or for moderate eccentricities of loading of the wagons. With regard to the Casewag vehicles, of which only five now exist, the Committee has asked for them to be modified to reduce their derailing tendencies.

General Summary

The Track Committee considered that cant should be provided to give equilibrium conditions at the speed selected as the basis for calculation.

The Committee's recommendations on limiting values which should be used in design are given in Table 4.

TABLE 4.—RECOMMENDED LIMITING VALUES

Maximum permissible cant	6 inches	
Maximum permissible rate of gain and loss of cant	2¼ inches per second	
Maximum permissible cant deficiency on Classes A and B lines	3½ inches	These figures to be reduced as cant increases
Maximum permissible cant deficiency on Classes C and D lines and in switch turnouts	2½ inches	
Maximum permissible rate of gain and loss of cant deficiency	2¼ inches per second	
Steepest permissible cant gradient	1 in 300	

Throughout the deliberations which eventually led to these decisions; members of the Committee emphasized that the value selected for each of the features was such as would permit of the deterioration of the condition of the track which normally occurs between occasions when the track has attention. Further, whilst these limiting values can be used with confidence, the Committee intends that they should only be used where made necessary by space restrictions, and the Committee would prefer the designer to use, where possible, values of rate of gain and loss of cant and of cant deficiency not exceeding 1½ inches per second, and to design transitions in which the cant gradients do not exceed 1 in 450. Transitions conforming to these figures will be 1½ times as long as those designed on the limiting values.

ACKNOWLEDGEMENTS

The Author thanks Mr J. C. L. Train, M.C., M.I.C.E., Member of the Railway Executive, and Mr T. M. Herbert, M.A., Director of Research, Railway Executive, for permission to publish the information in Part I of the Paper, and he gratefully acknowledges the suggestions put forward by members of the Track Committee; the help given by Mr W. M. Bond and his staff, Civil Engineers Department, London Midland Region Headquarters; the work on the track so willingly arranged by Mr J. Cunningham, District Engineer, North Wales, and done by his staff; and the enthusiastic assistance from the staff of the Railway Executive Research Department at Derby.

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Part II.—Application of the Experimental Data to Design

by

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INTRODUCTION

THE aspect of curve design dealt with in this part of the Paper is the relation between the form of the curve and the speed of trains. Design, in this sense, applies both to the planning of curves on new lines of railway and to the improvement of existing curves. It covers also the determination of permissible speeds on curves even where no improvement is possible. Methods of re-aligning curves are outside this aspect of the subject and are not discussed.

As mentioned in Part I, the methods of the former Railway Companies of Great Britain in the design of curves were not uniform. The advantage of uniform practice, particularly in the assessment of permissible speeds, is apparent, and the Railway Executive has therefore approved for general use certain principles and rules for curve design, the development and application of which are here explained. In the design of curves it is not possible to work to fixed rules in every respect. Conditions differ so widely that liberty is necessarily given to designers in the selection of suitable values, provided that they are within the specified limits. What has been laid down is therefore a code of practice, embodying certain limiting rules, rather than precise instructions to cover every feature of design.

To indicate the range of the subject two extreme cases may first be mentioned. On new lines of railway where there is no restriction of space and where, in consequence, a free choice of curvature and cant may be made, it is possible to design a curve for any required speed. On existing lines, on the other hand, cases are sometimes met where restricted clearances preclude any appreciable improvement of curvature or cant, and the problem is then to determine the maximum speed which the existing conditions will allow.

Neither of these cases is of common occurrence on British Railways. Most problems of curve design are concerned with curves where some adjustments of curvature and cant are possible, and where the designer's task is to determine the most suitable conditions in relation to the required or the permissible speed.

All these cases fall within the scope of the rules given in this Paper. The rules are intended primarily for curves with transitions; their application is later extended to the less usual case of curves without transitions, and to the more complicated conditions that occur in switch-and-crossing work. The rules are based on the relation between curvature, cant, and speed as derived from the principles of dynamics. They incorporate the limiting values of cant and of deficiency of cant, and of their rate of change, in accordance with the conclusions reached in Part I of the Paper.

Graphs based on the formulae are also provided. By their use the selection of suitable values for the several variables involved is greatly facilitated, and it will be found that most problems of curve design can be more readily solved by the graphs than by other means.

Derivation of Rules

The symbols used in deriving the formulae are as follows :—

v denotes the speed of the train in feet per second.
V ,, ,, ,, ,, ,, ,, in miles per hour.

V_e denotes equilibrium speed in miles per hour.
V_m ,, maximum permissible speed in miles per hour.
r ,, radius of curve in feet.
R ,, ,, ,, ,, in chains.
e_a ,, actual cant in feet.
E ,, ,, ,, in inches.
d ,, deficiency of cant in feet.
D ,, ,, ,, ,, in inches.
s ,, distance between centres of rails in feet.
g ,, acceleration due to gravity in feet per second per second.
W ,, weight of vehicle in pounds.
L ,, length of transition curve in feet.

While the principles stated below are true for track of any gauge, the practical formulae have been deduced for track of 4-foot-8½-inch gauge, the value of s being 4 feet 11¼ inches.

Conditions will be considered separately on the two parts of the curve, namely, the circular part and the transition. The rules given for the transition at the beginning of the curve will also apply to the transition at the end of the curve.

Circular Curve

1. Equilibrium Speed

When the speed of a vehicle on a canted curve is such that the resultant of the centrifugal force and the weight of the vehicle is normal to the canted track, "equilibrium conditions" are said to obtain and the speed is known as the "equilibrium speed." Under these conditions the weight is equally divided between the two rails, and the centrifugal force is balanced by the horizontal component of the normal reaction of the track. Since this component is proportional to the cant, the centrifugal force is said to be balanced by the cant.

In *Fig. 13*, the resultant F of the centrifugal force $\frac{Wv^2}{gr}$ and the weight W acts normally to the canted track AB. Owing to the very small angle that the canted track makes with the horizontal, AC may be considered equal to AB; that is, AC $= s$.

Hence $$\frac{Wv^2}{gr}\cdot\frac{1}{W}=\frac{e_a}{s}$$

And the cant, $$e_a=\frac{sv^2}{gr}\text{ feet.}$$

In practical units,

the cant, $$E=\frac{0{\cdot}06\,V_e^{\,2}}{R}\text{ inches}$$

and the equilibrium speed, $$V_e=4{\cdot}08\sqrt{RE}\text{ miles per hour.}$$

2. *Speed in Excess of Equilibrium Speed; Maximum Permissible Speed*

If the speed of the vehicle exceeds the equilibrium speed the centrifugal force is increased and the resultant F deviates outwards from the centre-line of the track, as shown in *Fig. 14*. If the actual cant e_a were increased by an amount d to a total of $e_a + d$, so that the resultant would again act normally to the canted track, equilibrium conditions would be restored, and then, as before, the total cant:

$$e_a + d = \frac{sv^2}{gr}$$

In practical units,

$$E + D = \frac{0{\cdot}06V^2}{R}$$

Fig. 13

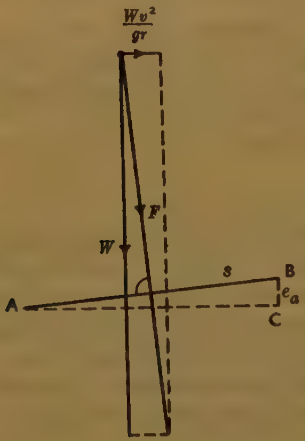
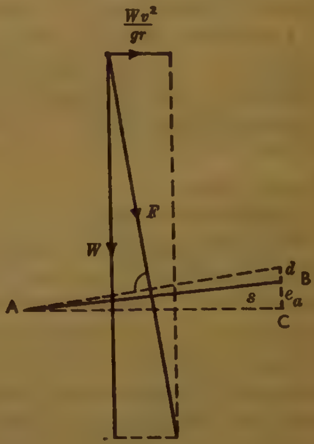


Fig. 14



If the increased cant is not provided and the actual cant remains at E, then D is the deficiency of cant. If D is the maximum allowable deficiency of cant, then the maximum permissible speed is given by:

$$V_m = 4{\cdot}08\sqrt{R\,(E + D)}$$

It will be evident that only a part of the centrifugal force is balanced by cant in this case, and that the deficiency of cant is proportional to the unbalanced part. It is proportional also to the increase of radial acceleration associated with the unbalanced centrifugal force.

The maximum allowable deficiency of cant for any value of actual cant, as based on the conclusions given in Part I, is shown in Table 5.

Fig. 15 (a), Plate 2, has been constructed from the formulae for equilibrium and maximum permissible speeds given above. It gives the equilibrium speed for any radius of curve with any value of cant. It also

TABLE 5.—MAXIMUM ALLOWABLE DEFICIENCY OF CANT

Class of line	Actual cant, E: inches							Maximum deficiency of cant D expressed in terms of E inches
	0	1	2	3	4	5	6	
A and B . .	$3\frac{1}{2}$	$3\frac{1}{4}$	3	$2\frac{3}{4}$	$2\frac{1}{2}$	$2\frac{1}{4}$	2	$3\frac{1}{2}-\frac{1}{4}E$
C and D and turnouts .	$2\frac{1}{2}$	$2\frac{1}{4}$	2	$1\frac{3}{4}$	$1\frac{1}{2}$	$1\frac{1}{4}$	1	$2\frac{1}{2}-\frac{1}{4}E$

Definitions of the classes of line will be found in the Appendix, p. 541.

gives the maximum permissible speed for any radius of curve when the maximum allowable deficiency of cant is added to the actual cant.

The figures in Table 5 are plotted in Fig. 15 (*c*), Plate 2. Values of $E + D$ are plotted in Fig. 15 (*b*). For intermediate values of cant the appropriate deficiency of cant can be interpolated, or calculated from the formulae given in the table, or read from Fig. 15 (*c*).

Transition Curves

1. The Purpose and Form of Transition Curves

The purpose of a transition curve is to allow the centrifugal force to be gained gradually and at a comfortable rate. The ideal transition between straight track and a circular curve is one in which the curvature increases uniformly from zero at the tangent with the straight to its full value on reaching the circular curve. The cant and the deficiency of cant, which are proportional respectively to the balanced and the unbalanced parts of the centrifugal force, should also increase uniformly in the same distance, so that at any point on the transition their relation to the curvature will be constant. The length of the transition should be such that at maximum speed the rates of increase of cant and of deficiency of cant are not greater than those specified in Part I, namely, $2\frac{1}{4}$ inches per second in each case.

The condition that curvature should increase uniformly will be fulfilled with a close degree of accuracy by providing a transition curve of cubic parabola form. The equation of the cubic parabola is:

$$y = kx^3$$

and the curvature at any point is approximately:

$$\frac{d^2y}{dx^2} = 6kx$$

When $x = L$ the full length of the transition, the curvature will be $\frac{1}{r}$ where r is the radius of the circular curve. Hence $\frac{1}{r} = 6kL$ and $k = \frac{1}{6rL}$. The equation of the curve is therefore $y = \frac{x^3}{6rL}$

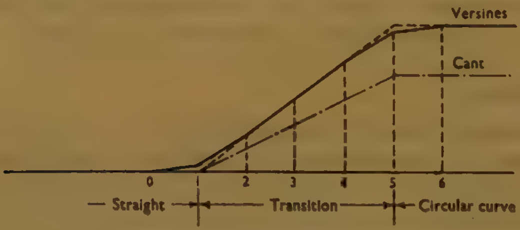
The curvature at any point distant x from the origin is then:

$$\frac{d^2y}{dx^2} = \frac{x}{rL}$$

and, since r and L are constants, the curvature is in direct proportion to the distance x. It follows that the versines of the transition curve will increase uniformly as shown in *Fig. 16*. Slight adjustments are required at the tangent points on account of the chord extending on to the straight or on to the circular curve. Thus at point 1 the versine, instead of being zero, is about one-sixth of the versine at point 2. A similar adjustment is shown at point 5 in the diagram. *Fig. 16* also shows a typical cant gradient.

The same principles apply to the transition at the other end of the curve.

Fig. 16



FORM OF TRANSITION CURVE

2. *Minimum Length of Transition Curve*

If the cant and the deficiency of cant increase uniformly from zero to their full value in the length of the transition curve, their rate of increase will be constant, provided that the speed of the train remains constant. If the time taken to run through a transition curve of length L feet at a speed of v feet per second is t seconds, and the cant, E inches, is to be gained at a rate of 2¼ inches per second, then:

$$L = vt = \frac{vE}{2\frac{1}{4}} \text{ feet}$$

Similarly, for deficiency conditions:

$$L = vt = \frac{vD}{2\frac{1}{4}} \text{ feet}$$

In practical units, the minimum length of transition is:

$$L = 0{\cdot}65\, EV_m, \text{ or } L = 0{\cdot}65\, DV_m \text{ feet;}$$

the greater of the two lengths being used.

The above formula for the minimum length of transition based on deficiency of cant applies for all speeds. The formula based on cant applies only to speeds above 38·5 miles per hour. Below this speed it will be found that the formula $L = 0{\cdot}65\, EV_m$ gives a cant gradient steeper than 1 in 300, the limiting gradient specified in Part I. Below this speed therefore the minimum length of transition is given by:

$$L = 25E \text{ feet}$$

which represents a cant gradient of 1 in 300, or 1 inch in 25 feet.

The above formulae have been used in constructing Fig. 15 (*d*), Plate 2, which therefore gives the minimum length of transition for any maximum speed, with any value of cant or of deficiency of cant. It is emphasized that the length of transition obtained by the above rules is the absolute minimum, and that, where space permits, a longer transition should be provided. A transition at least 1½ times the length obtained by the formulae or from Fig. 15 (*d*), Plate 2, should be provided where possible.

3. *Transition Curve of Restricted Length*

Where space does not permit the use of the minimum length of transition specified above the longest transition possible should be used, and a lower cant or a lower deficiency of cant, or both, should be selected; this will have the effect of reducing the maximum speed. The values of cant and of deficiency may be obtained by equating the expressions for maximum speed on the transition and on the circular curve, thus:

$$V_m = 4{\cdot}08\sqrt{R(E + D)} = \frac{L}{0{\cdot}65E}$$

Substituting $(3\tfrac{1}{2} - \tfrac{1}{4}E)$ for D in the case of class A and B lines, the required cant is obtained from the equation:

$$7{\cdot}03(\tfrac{3}{4}E^3 + 3\tfrac{1}{2}E^2) = \frac{L^2}{R}$$

The corresponding equation for class C and D lines is:

$$7{\cdot}03(\tfrac{3}{4}E^3 + 2\tfrac{1}{2}E^2) = \frac{L^2}{R}$$

The cant having been obtained, the maximum allowable deficiency of cant can be found from Table 5 or Fig. 15 (*c*), Plate 2. The maximum speed can then be calculated from the above formulae or read from Fig. 15 (*a*), Plate 2. If, however, the maximum allowable deficiency is greater than the cant, it is desirable to restrict it to the same value as the cant, since maximum speed in this case will be obtained when the cant equals the deficiency of cant.

These rather troublesome calculations can be avoided by the use of Fig. 15 (*e*), Plate 2, which has been constructed from the above formulae. For a given radius and length of transition and for a given classification of line the graph indicates the values of cant and of deficiency of cant which will give maximum speed. In using the graph the radius and length of transition are first found on the right-hand side. Reading acrcss to the class of line, the cant is found at the foot of the upper left-hand graph. The appropriate deficiency of cant can be read on the left. The maximum permissible speed is found in the lower left-hand graph, from the cant and the length of transition. When, in passing from the right to the left-hand graph, the dotted part of the classification line is reached, maximum speed is obtained when cant and deficiency are equal. Where the classification line is shown in full, cant exceeds deficiency. The examples given later will make the use of the graph clear.

Fig. 15 (*e*), Plate 2, does not cover cases where the length of transition is governed by the maximum cant gradient of 1 in 300; that is, where $L = 25E$, and where maximum speeds are consequently less than 38·5 miles per hour. Such cases can be directly calculated from the known length transition of and the maximum amount of cant which that length allows.

AMOUNT OF CANT TO BE PROVIDED

No precise rules can be given for the amount of cant to be provided on a curve. Generally it should be the equilibrium cant for a selected speed, which is then called the equilibrium speed. The selection of this speed is a matter for careful judgement, taking into consideration the local circumstances and the nature and speed of the various kinds of traffic using the line. Formulae have been suggested for arriving at the mean speed for which cant is to be provided, but they do not take sufficiently into account all the factors involved, and are therefore liable to be misleading.

Among the principal factors to be considered are :—

1. The speed limit for the route. Curves should be designed and cant should be provided so that the speed limit for the route can be maintained wherever possible and restrictions can be avoided.
2. The proximity of permanent speed restrictions, junctions, and stopping places, which may influence the speed on the curve under consideration.
3. Up-gradients, which may reduce the speed of freight and slow passenger-trains without appreciably affecting the speed of the faster trains.
4. Down-gradients, which may provide valuable recovery time on a difficult route.
5. The relative importance and the timings of the various kinds of traffic using the line.

On a curve where all trains travel at about the same speed, equilibrium cant should be provided to suit this speed.

On a curve carrying mixed traffic running at different speeds, cant should be provided to suit a mean speed. This will usually be the average speed of ordinary express passenger-trains, but it should be so selected as to avoid, if possible, imposing a speed restriction on the fastest trains.

On lines chiefly used by slow freight traffic it may be necessary to limit cant so as to avoid excessive wear of the low rail, even though this may result in a speed restriction on faster trains.

There are some cases where cant can be directly determined, as, for instance, where it is limited on account of clearances, or of transitions of restricted length, or of switch and crossing work.

It is not necessary always to use the maximum allowable deficiency of cant; nor is it always desirable, for it may lead in time to excessive wear of the outer rail and to difficulties of maintenance. Maximum allowable deficiency should generally be reserved for places where the cant is limited, or where it is desirable to avoid imposing a restriction on the faster trains.

As stated in Part I, cant should never exceed 6 inches. Values of cant above 6 inches shown in Fig. 15 (*a*), Plate 2, represent cant plus deficiency, and not cant to be actually provided on the track.

Cant is usually specified and indicated on the track to the nearest $\frac{1}{4}$ inch. On transition curves the indications need not necessarily coincide with the chord survey points.

EXAMPLES OF THE USE OF THE GRAPHS FOR SIMPLE CURVES

(1) A 50-chain curve on a class A line is to be provided with equilibrium cant for 50 miles per hour; it is desired to find the cant, the maximum permissible speed and the desirable length of transition.

From Fig. 15 (*a*), Plate 2, the cant for 50 miles per hour on a curve of 50 chains radius is read as 3 inches. From Fig. 15 (*c*) the maximum allowable deficiency for 3 inches cant is read as $2\frac{3}{4}$ inches. The maximum permissible speed is then read from Fig. 15 (*a*) for 50 chains radius and a total of $3 + 2\frac{3}{4}$ inches cant, and is 69 miles per hour.

The cant, being greater than the deficiency, governs the length of the transition, the minimum length of which for 69 miles per hour and 3 inches cant is read from Fig. 15 (*d*) as 134 feet. The desirable length is at least $1\frac{1}{2}$ times $134 = 201$ feet.

(2) A 60-chain curve on a class A line is to be canted for a maximum speed of 80 miles per hour; it is desired to find the cant and the desirable length of transition.

From Fig. 15 (*a*), Plate 2, cant plus deficiency for 60 chains radius and 80 miles per hour is read as 6·4 inches. From Fig. 15 (*b*) it is found that the cant will be about 4 inches; the deficiency will therefore be 2·4 inches. From Fig. 15 (*d*), it is seen that the minimum length of transition for a speed of 80 miles per hour and 4 inches cant will be 208 feet. The desirable length will be 312 feet.

(3) On a 35-chain curve on a class B line no cant can be provided; it is desired to find the maximum permissible speed and the minimum length of transition.

From Fig. 15 (*c*), Plate 2, the deficiency for no cant is read as $3\frac{1}{2}$ inches. From Fig. 15 (*a*) the maximum speed for 35 chains radius and $3\frac{1}{2}$ inches deficiency is found to be 45 miles per hour. From Fig. 15 (*d*), the minimum length of transition for 45 miles per hour and $3\frac{1}{2}$ inches deficiency is found to be 102 feet.

(4) On a 30-chain curve on a class B line the length available for a transition is only 120 feet; it is desired to find the cant, deficiency, and maximum speed.

Using Fig. 15 (*e*), Plate 2, reading from 30 chains on the right-hand side up to the 120 feet transition line, and thence across to the A and B classification line, the cant is read at the foot of the upper left-hand graph as 3·35 inches. The deficiency is also read on the left as 2·65 inches. The maximum speed for 3·35 inches cant and 120 feet transition is read in the lower left-hand graph as 55 miles per hour.

(5) On a 25-chain curve on a class A line the length of transition is limited to 66 feet; it is desired to find the cant, deficiency, and maximum speed.

From Fig. 15 (*e*), Plate 2, the cant for 25 chains radius and a 66 feet transition is read as 2·3 inches. This being on the dotted part of the classification line the maximum speed will be obtained when cant equals deficiency. Maximum speed for 2·3 inches cant, or an equal amount of deficiency, with a 66 feet transition, is read as 44 miles per hour.

(6) On a 10-chain curve on a class D line the cant is to be limited to 2 inches; it is desired to find the deficiency, the maximum speed, and the minimum length of transition.

From Fig. 15 (*c*), Plate 2, it is found that the maximum deficiency will be 2 inches, and from Fig. 15 (*a*), that the maximum speed will be 25·8 miles per hour. Since the speed is low, the limiting cant gradient of 1 in 300 may have to be considered. From Fig. 15 (*d*), the minimum length of transition for 2 inches deficiency is read as 33·6 feet, but the minimum length for 2 inches cant is 50 feet, and this length at least must be provided.

COMPOUND CURVES

It is desirable to design the two circular parts of a compound curve for a uniform maximum speed. This involves the selection of suitable values for the cant and the deficiency of cant. The minimum length of transition connecting the curves will depend on these values, and will be given by one of the following formulae, the greatest of the three lengths obtained being adopted,

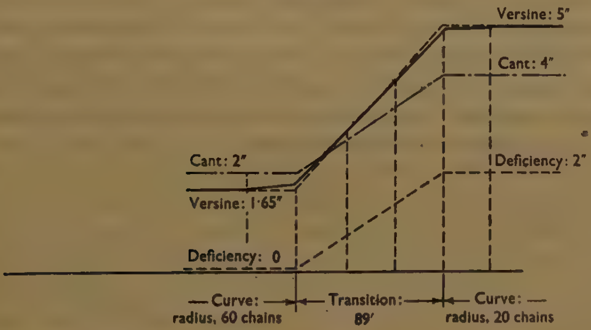
$$L = 0{\cdot}65(E_1 - E_2)V_m$$
$$L = 0{\cdot}65(D_1 - D_2)V_m$$
$$L = 25(E_1 - E_2)$$

Where space does not permit a full length transition, the cants and deficiencies and the maximum speed should be appropriately reduced. Where no transition can be provided, the case should be dealt with as described under the heading "Curves without Transitions."

Example: A compound curve on a class A line has radii of 60 and 20 chains; it is desired to find the cant and deficiency, and the length of transition required for a maximum speed of 45 miles per hour.

From Fig. 15 (*a*), Plate 2, $E + D$ for 60 chains radius and 45 miles per hour is read as 2 inches; and $E + D$ for 20 chains radius and 45 miles per hour is 6 inches. Suitable values would be $E = 2$ inches and $D = 0$ for the 60-chain curve, and $E = 4$ inches and $D = 2$ inches for the 20-chain curve, as these values give the same change of cant as of deficiency, and therefore the same minimum length of transition, $L = 0{\cdot}65\ (4 - 2)\ 45 =$ 59 feet. If space permits it would be desirable to provide a transition not less than $1\frac{1}{2}$ times 59 = 88·5 feet. *Fig. 17* shows this length of transition and the way in which the versines, the cant, and the deficiency change.

Fig. 17



Compound Curve
($V_M = 45$ miles per hour)

The versines are calculated on a 66-foot chord.

Reverse Curves

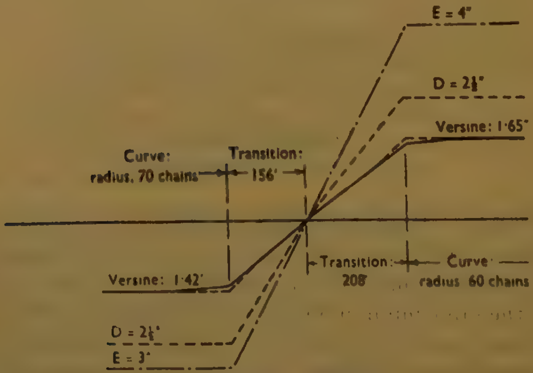
In reverse curves the end of each curve should have a transition of the standard form and of length to suit the values of cant and of deficiency selected. As in the case of compound curves, these values should be chosen so that the maximum speed on the two curves will be uniform. Where a full-length transition cannot be provided these values should be appropriately reduced and a lower maximum speed will be obtained. Reverse curves without transitions are dealt with later in the Paper.

It is not desirable to interpose a short length of straight track between reverse transitions if this can be avoided. It is found that steadier running is obtained when the gradual rotation of a vehicle due to reversal of cant is not interrupted. Similarly, it is undesirable to interrupt the gradual reversal of side thrust due to deficiency of cant. If necessary, the transitions should be slightly lengthened to eliminate a short length of straight if one exists.

Example: Reverse curves on a class A line have radii of 60 and 70 chains; it is desired to find the cants and deficiencies and the minimum lengths of the transitions for a maximum speed of 80 miles per hour.

From Fig. 15 (*a*), Plate 2, $E + D$ for 60 chains radius and 80 miles per hour is $6\frac{1}{2}$ inches and $E + D$ for 70 chains radius and 80 miles per hour is $5\frac{1}{2}$ inches. A suitable choice of cants and deficiencies would be $E = 4$ inches and $D = 2\frac{1}{2}$ inches for the 60-chain curve, and $E = 3$ inches and $D = 2\frac{1}{2}$ inches for the 70-chain curve. From Fig. 15 (*d*), Plate 2, the minimum length of transition for the 60-chain curve with 4 inches cant is 208 feet, and for the 70-chain curve with 3 inches cant is 156 feet. *Fig. 18* illustrates this example.

Fig. 18



Reverse Curves
($V_M = 80$ miles per hour)

Curves Without Transitions

Curves without transitions are seldom found on plain line at the present time. They are more common, however, in junctions and connexions, and particularly in crossovers. It is necessary, therefore, to consider some means of assessing permissible speeds in such cases.

The foregoing rules are not directly applicable to curves without transitions, but they can be made to apply by adopting the principle of the "virtual transition." It must be remembered that the rules are based on the comfort of passengers travelling in bogie coaches. Considering the motion of such a coach running at uniform speed directly from the straight on to a curve, it will be seen that the whole vehicle begins to acquire radial acceleration when the leading bogie passes the tangent point. It continues to gain radial acceleration until the trailing bogie reaches this point, after which the vehicle is subject to uniform circular motion. The change of motion is thus completed in a distance equal to the length between the centres of bogies. The shortest distance between the bogie centres of the British standard coaches, namely 40 feet, has therefore been accepted as the length of the virtual transition.

The maximum permissible speed at the virtual transition may be obtained from the formulae previously used :—

$$L = 0{\cdot}65EV_m, \quad L = 0{\cdot}65DV_m, \quad \text{or } L = 25E,$$

when L is 40 feet and E and D are known or are suitably selected. The deficiency of cant is considered as being gained within the length of 40 feet, commencing on the straight 20 feet before the tangent point. The cant will normally be gained in the same distance, the run-up commencing at the same point. In order that the maximum cant gradient of 1 in 300 shall not be exceeded, the amount of cant should generally be limited to $40 \div 25 = 1{\cdot}6$ inches. In special cases where it is necessary to provide a greater amount of cant it may be run up in 80 feet, commencing on the straight 60 feet before the tangent point.

The cant and the deficiency of cant should be selected so as to give a uniform maximum speed on the curve and on the virtual transition; or, if more accurate values are desired, they may be obtained by equating the two expressions for maximum speed, as shown below. Care should be taken that the limits specified for the amount and the rate of change of these values are not exceeded.

The principle of the virtual transition holds good also for compound and reverse curves. It is necessary to consider only the case of compound curves when the radii are widely different; the change of cant or of deficiency is not generally large enough to reduce the speed on the virtual transition below that for the sharper of the two curves. On reverse curves, however, the sum of the cants or of the deficiencies on the two curves has to be considered at the reverse, and the maximum speed on the virtual transition usually governs the overall maximum permissible speed. The application of the principles to a few cases of common occurrence is shown below; the numerical examples are used later in the tables of speeds for crossovers.

Curve tangential to the straight; no cant

Equating the maximum speeds for the curve and the virtual transition:

$$V_m = 4{\cdot}08\sqrt{RD} = \frac{40}{0{\cdot}65D}$$

Hence,
$$D = \frac{6{\cdot}10}{\sqrt[3]{R}}$$

If R is 21 chains, D by the above formula will be 2·21 inches, and V_m will be 27·8 miles per hour.

Reverse Curves; No Cant

There is often a short length of straight between reverse curves. If it is more than 40 feet in length the two curves are considered separately as curves tangential to the straight. If it is less than 40 feet, change of motion of a vehicle from one curve to the other takes place in a distance of $A + 40$ feet, where A is the length of straight. Let R_1 and R_2 be the radii of the two curves and D_1 and D_2 their deficiencies of cant. Equating maximum speeds on the two curves and on the virtual transition:

$$V_m = 4{\cdot}08\sqrt{R_1D_1} = 4{\cdot}08\sqrt{R_2D_2} = \frac{A+40}{0{\cdot}65(D_1+D_2)}$$

Hence,
$$D_1 = \sqrt[3]{\frac{R_2{}^2(A+40)^2}{7{\cdot}04R_1(R_1+R_2)^2}}$$

V_m may then be calculated or read from *Fig. 15* (*a*). If R_1 is $25\frac{1}{2}$ chains, R_2 $23\frac{3}{4}$ chains, and A is 37 feet 10 inches, D_1 by the above formula will be 1·99 inches, and V_m will be 29·0 miles per hour.

Where reverse curves have the same radii, the deficiencies will be equal, and the above formula reduces to:

$$D = \sqrt[3]{\frac{(A+40)^2}{28{\cdot}16R}}.$$

Hence,
$$V_m = 2{\cdot}34\sqrt[3]{R(A+40)}.$$

These formulae apply to single crossovers with like turnouts, between straight and parallel main lines, where the radius of the turnout is not less than $14\frac{1}{2}$ chains. Below this radius the maximum deficiency of cant of $2\frac{1}{2}$ inches governs the speed. If R is 21 chains and A is 20 feet 10 inches, D by the above formula will be 1·84 inches, and V_m will be 25·4 miles per hour.

The above formulae will apply to "immediate reverses," that is, where there is no straight between the two curves, by putting A as zero. In the particular case where the radii are equal and there is no cant,

$$V_m = 8{\cdot}0\sqrt[3]{R}.$$

TURNOUTS, CROSSOVERS, AND JUNCTIONS

Turnouts

Much consideration has been given to the most convenient way of determining the maximum permissible speed through turnouts. The conditions resemble those on a curve without a transition, with the additional complications that there is a sudden divergence owing to the angle which the switch makes with the stock rail, that the switch is straight throughout the planed length and curved beyond the planing; and also that in some cases the turnout curve is sharper than the switch curve. Moreover, the severity of the divergence at the switch, and the radii of the switch and turnout curves, vary according to the degree and direction of curvature of the main line.

The radius of the turnout curve affords the most ready measure of these varying conditions and of the severity of the deviation from the main line. It is therefore convenient to base the maximum permissible speed on the radius of the turnout curve, and to reduce the allowable deficiency of cant, as compared with plain line, so as to compensate for the sudden divergence at the switch. The reduced deficiency should be applied to the diverging or converging lines throughout a junction or group of connexions in order to allow for the disturbing effect of crossings and check-rails on sharp curves. It need not be applied to curves of greater radius than 40 chains as these effects are less noticeable on flatter curves.

The maximum speed on turnouts is therefore given by the formula for simple curves:

$$V_m = 4{\cdot}08\sqrt{R(E + D)}$$

where R is the radius of the turnout curve in chains, and E and D are the cant and deficiency of cant, D having the values for turnouts shown in Table 6.

Example: In a turnout consisting of D-switches and a 1-in-12 crossing, diverging from a straight main line and laid without cant, the radius of the turnout curve is 21 chains. The maximum permissible speed as calculated by the above formula will be 29·5 miles per hour. This result may also be obtained from Fig. 15 (*a*), Plate 2.

Turnouts with Negative Cant

Cases where the inner rail of a turnout is elevated above the outer rail occur sometimes in turnouts from a curved main line of contrary flexure, where the cant is applied to suit the main line. The cant on the turnout is then known as "negative cant." The rule for maximum speed on turnouts will apply provided that the cant E_n is given a negative value: thus,

$$V_m = 4{\cdot}08\sqrt{R(-E_n + D)}$$

D will have the same value for negative cant as for positive cant, as shown in Table 6.

Example: If the radius of the turnout curve is 21 chains and there is 1 inch of negative cant, the allowable deficiency, from Table 5, is 2·25 inches. The maximum speed will then be:

$$V_m = 4{\cdot}08\sqrt{21(-1{\cdot}0 + 2{\cdot}25)} = 20{\cdot}9 \text{ miles per hour.}$$

This result may also be obtained from Fig. 15 (*a*), Plate 2.

If E_n in the above formula is equal to or greater than D, the formula does not hold good, and a speed of 10 miles per hour may be allowed in such cases; this also applies when $(-E_n + D)$ is positive but gives rise to speeds less than 10 miles per hour.

It is undesirable for negative cant to exceed 3 inches at switches and common crossings, and 2½ inches at obtuse crossings.

Single Crossovers

The maximum speed for each turnout should be obtained from the formulae stated above. The maximum speed on the reverse should be calculated by the rules given for reverse curves without transitions, allowance being made for the length of straight track between the crossings. The overall maximum speed will be the lowest of the values obtained.

Table 6 shows the maximum speeds for the turnouts and for the reverse for some typical crossovers between straight main lines, with various intervals between tracks, and laid without cant. The speeds for the "D and 12" crossover have already been worked out as examples in the previous sections; the other speeds have been arrived at in the same manner. Speeds to the right of the thick line remain the same for any particular crossover because the distance between the crossings is 40 feet or more, and they are therefore dealt with as curves tangential to the straight and not as reverse curves.

TABLE 6.—MAXIMUM SPEEDS FOR SINGLE CROSSOVERS

Turnout. Switch and crossing	Radius of turnout curve: chains	Speed on turnout: miles per hour	Speed on reverse (miles per hour) for various distances between lines (running edges)						Speed restriction: miles per hour
			6′–5½″	7′–5½″	8′–5½″	9′–5½″	10′–5½″	11′–5½″	
B & 8 .	9½	19·7	18·6	19·5	19·7	19·7	19·7	19·7	15
C & 10 .	14½	24·6	22·0	23·2	24·3	24·6	24·6	24·6	20
D & 12 .	21	29·5	25·4	26·9	27·8	27·8	27·8	27·8	25
E & 16 .	37	39·4	31·9	33·6	33·6	33·6	33·6	33·6	30
F & 20 .	58	58·1	38·2	39·0	39·0	39·0	39·0	39·0	35

If the main lines are curved, the radii and consequently the maximum speeds for the turnouts will differ. The deficiencies will also differ, that on the sharper curve being increased, while that on the flatter curve will be correspondingly reduced. The sum of the deficiencies will, however, remain the same if the turnouts are alike in size. Hence the total change of deficiency will be the same as for a crossover of the same dimensions between straight main lines, and the reverse speeds shown in the table above will hold good. The formula for maximum speed on the reverse

$$V_m = 2{\cdot}34\sqrt[3]{R(A+40)}$$

will apply if R is the equivalent radius of either turnout curve from a straight main line, as calculated by the well-known rule.

Through Crossovers

The rules already given are applicable to the various parts of through crossovers crossing one or more lines. The overall maximum speed will be the lowest of the values obtained for the several parts. Some examples of typical uncanted crossovers between pairs of straight and parallel main lines are given in Table 7.

Double Junctions

The speeds given in Table 7 will apply to double junctions on straight main lines up to the obtuse crossings of the diamond. Beyond this point, conditions may vary widely, but the appropriate rules can be applied.

Canted Curves through Turnouts, Junctions, and other Connexions

An appreciably higher speed can usually be obtained by canting curves in turnouts and other connexions. The use of two-level chairs or base-plates enables cant to be provided to a limited extent on the diverging line, independently of the main line. It is a particular advantage to use them in turnouts and junctions of contrary flexure to the main line, and often avoids severe speed restrictions due to negative cant.

The foregoing rules can be applied to the several parts of canted connexions and the maximum speed for the whole of the diverging line will be the lowest of the values obtained.

SPEED LIMITS AND RESTRICTIONS

A Speed Limit is the maximum speed allowed on a particular line or branch or on sections thereof.

A Speed Restriction is a limitation of speed on a stretch of line where the speed must be below the general speed limit. Speed restrictions are specified in steps of 5 miles per hour. The nearest multiple of 5 below the calculated maximum permissible speed should be taken as the speed restriction, unless the calculated figure is itself a multiple of 5. It will be noted that the appropriate speed restrictions have been shown for each of the crossovers in Tables 6 and 7.

GENERAL CONSIDERATIONS IN CURVE DESIGN

Under the heading "Amount of Cant to be Provided," attention was called to the need, when designing a particular curve, to consider the nature and speed of traffic using the line and the overall speed limit. Some stress has also been laid on the desirability of designing the various parts of a curve for a uniform maximum speed, so that the whole curve will be of balanced design.

These considerations are of equal importance when a series of curves is being adjusted, especially if they are on an important main line. If curves are treated individually, without reference to general conditions on the route, there is a liability that their resulting speed-values may be unequal. Frequent variations of speed limits or restrictions should be avoided, because they are difficult to observe and are apt to be disregarded.

Where gradients are easy and there are no other restrictive conditions, it is desirable to design for a uniform maximum speed, if not for the whole route, at least for a considerable section. Where, owing to the nature of the line, uniform speed cannot be maintained, the speeds attainable on various sections should be ascertained and, if practicable, curves should be designed to suit those speeds. Thus, for instance, on a long falling gradient curves should be designed to give traffic the benefit of a higher speed, and it should be remembered that the increased speed may continue for a long distance beyond the gradient.

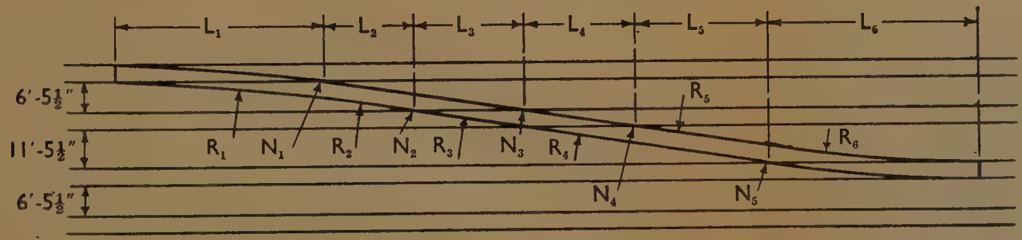
A general review of conditions on a line will indicate the curves which need radical treatment to suit the speeds attainable, and those where a minor adjustment will suffice. Work can then be concentrated on those improvements which will give the greatest benefit to traffic.

ACKNOWLEDGEMENTS

The Author wishes to thank the Railway Executive, and particularly Mr J. C. L. Train, M.C., M.I.C.E., the Member for Civil Engineering, for permission to present this part of the Paper and to use information and diagrams prepared for a booklet on the subject to be issued to the Civil Engineers' staff of British Railways.

The Paper is accompanied by fifteen sheets of diagrams, from which the Figures in the text have been prepared, and by the following Appendix.

TABLE 7.—SPEEDS FOR THROUGH CROSSOVERS WITH MAIN LINES STRAIGHT AND 11 FEET 5½ INCHES BETWEEN THE TWO PAIRS OF DOUBLE LINES

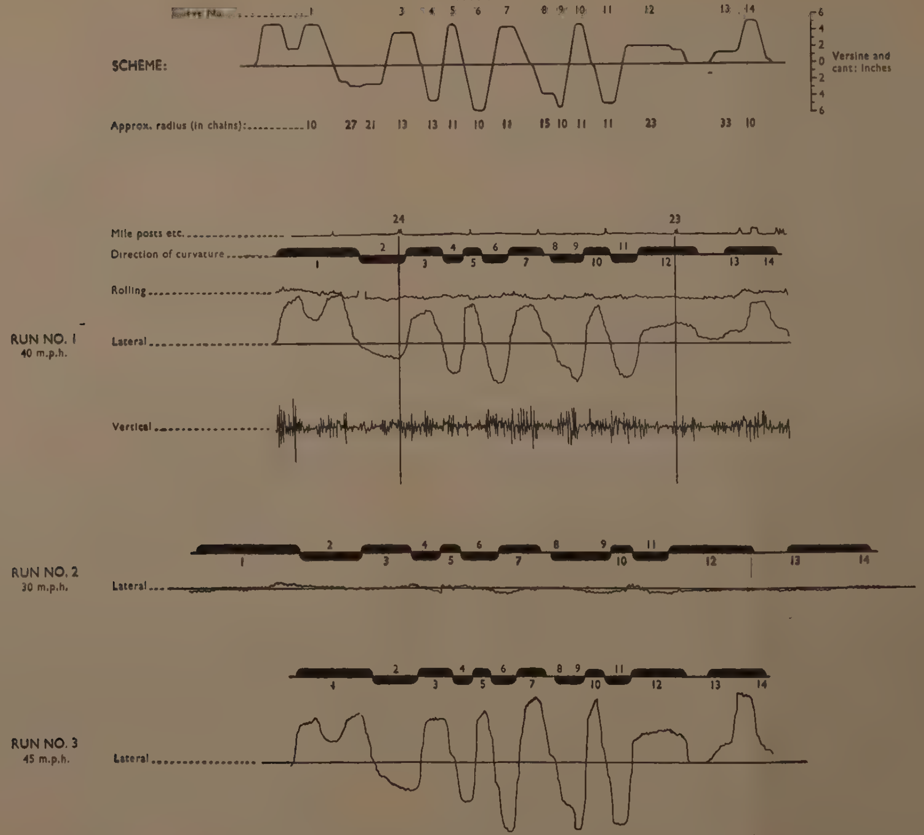


Crossing Lead Radius		L_1 R_1	N_1	L_2 R_2	N_2	L_3 R_3	N_3	L_4 R_4	N_4	L_5 R_5	N_5	L_6 R_6		Overall Length	Speed restriction miles per hour
S & C Lead Radius V_m	*D*	97′–0″ 21 30	12	18′–10″ 17½ 27	10	42′–0″ 25½ 29*	8	37′–10″ Straight	8	64′–5″ 23¾ 29*	12	97′–0″ 21 30	*D*	357′	25
S & C Lead Radius V_m	*D*	93′–3″ 16¾ 27	11	18′–1″ 30½ 36	10	42′–0″ 25½ 31*	8	37′–10″ Straight	8	62′–2″ 28 31*	11	93′–3″ 16¾ 27	*D*	346′	25
S & C Lead Radius V_m	*D*	97′–0″ 21 30	12	19″–10′ 41 41	11	49′–5″ 81 44†	10 *SD*	47′–3″ Straight	10	73′–4″ 67½ 41†	12	97′–0″ 21 30	*D*	384′	30
S & C Lead Radius V_m	*E*	129′–4″ 37 39	16	23′–10″ 17½ 27	12	45′–5″ 16½ 26*	8	37′–10″ Straight	8	71′–8″ 17½ 26*	16	129′–4″ 37 39	*E*	437′	25
S & C Lead Radius V_m	*E*	121′–11″ 26¾ 33	14	21′–4″ 17 27	11	43′–9″ 19½ 27*	8	37′–10″ Straight	8	68′–5″ 19½ 27*	14	121′–11″ 26¾ 33	*E*	415′	25
S & C Lead Radius V_m	*E*	129′–4″ 37 39	16	25′–11″ 45 43	14	60′–11″ 78 43†	12 *SD*	56′–7″ Straight	12	92′–4″ 67½ 41†	16	129′–4″ 37 39	*E*	494′	35

Note.—*SD* denotes Switch Diamond.
Lower deficiency for turnouts has been used in calculating V_m for curves R_1, R_2, and R_6.
* V_m calculated for reverse curves with intervening straight less than 40 feet.
† V_m calculated for curve tangential to straight without transition.

RECENT DEVELOPMENTS IN RAILWAY CURVE DESIGN

FIGS 1



RECORDS FROM HALLADE INSTRUMENT

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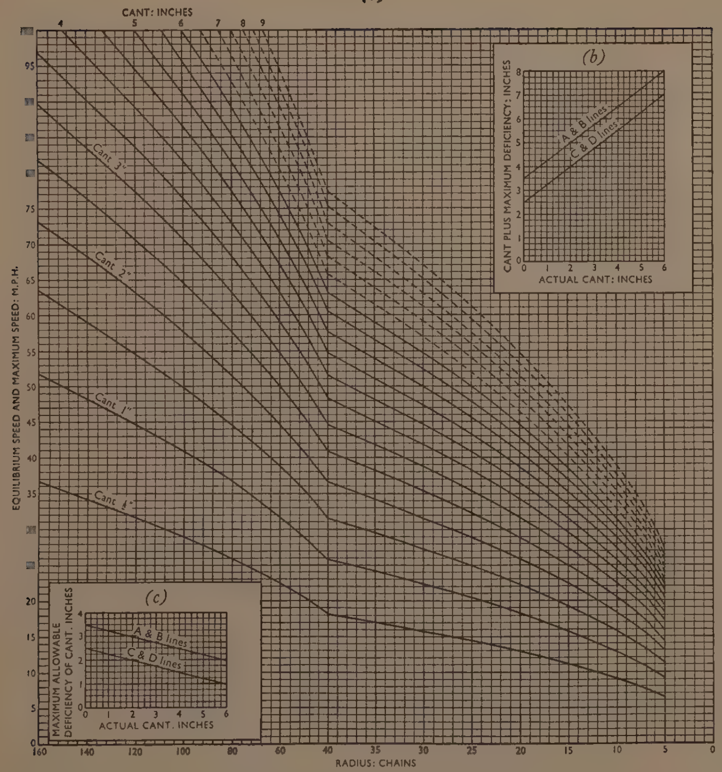
PLATE 2
RECENT DEVELOPMENTS IN
RAILWAY CURVE DESIGN

RECENT DEVELOPMENTS IN RAILWAY CURVE DESIGN

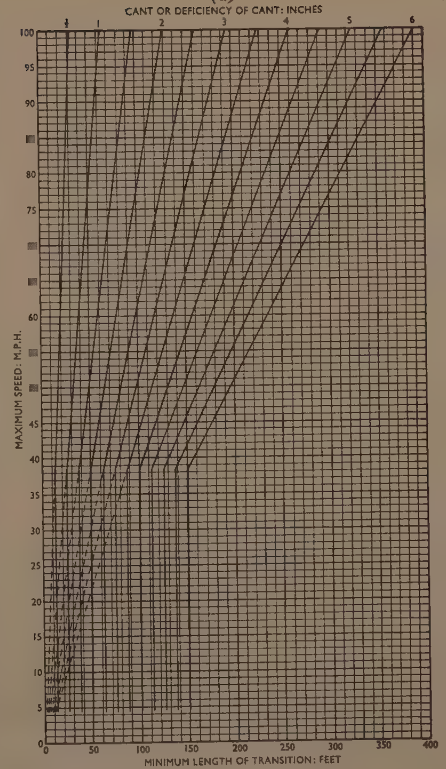
Figs 15

(a)



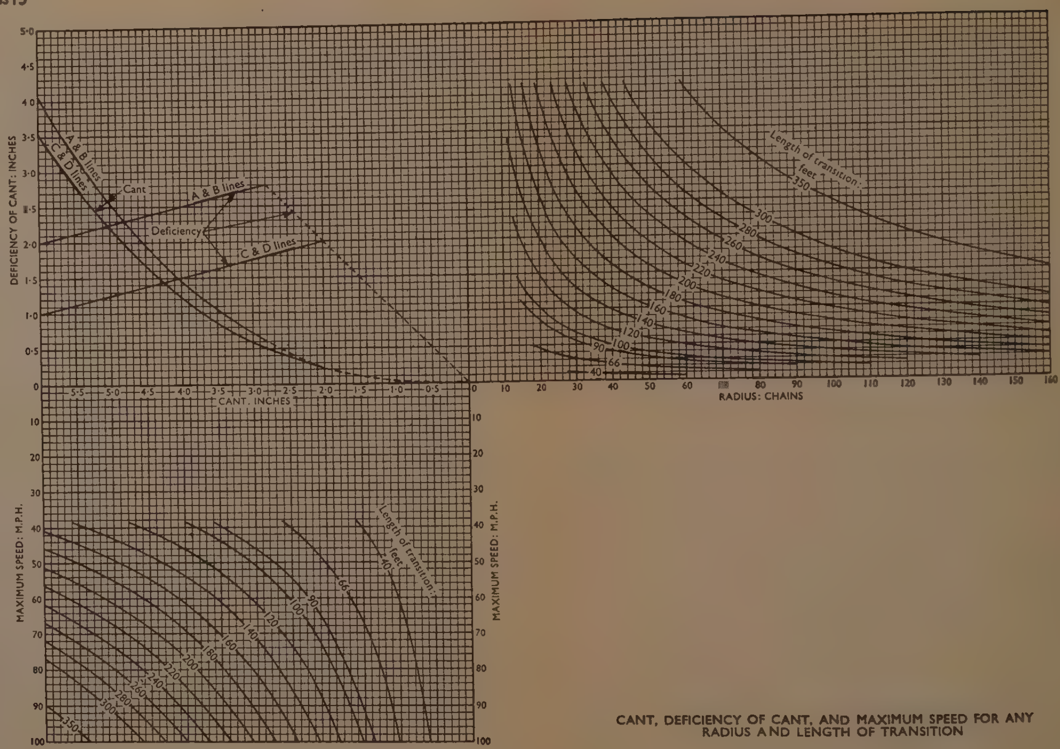
CANT AND SPEED FOR ANY RADIUS OF CURVE

(d)



MINIMUM LENGTH OF TRANSITION FOR ANY MAXIMUM SPEED AND ANY CANT OR DEFICIENCY OF CANT

(e)



CANT, DEFICIENCY OF CANT, AND MAXIMUM SPEED FOR ANY RADIUS AND LENGTH OF TRANSITION

J. C. LOACH and M. G. MAYCOCK

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APPENDIX

CLASSIFICATION OF LINES

The running lines of British Railways are classified according to their importance and the type, weight, and speed of the traffic which they carry, in accordance with the ruling of the Railway Executive shown in Table 8.

TABLE 8.—DEFINITION OF CLASSES OF LINE

Class of Line	Definition
A	Lines subject to speeds exceeding 60 miles per hour, over which twelve or more express passenger-trains operate regularly (winter timetable) per 24 hours.
B i	Lines subject to speeds of 60 miles per hour and over, but not qualified to be classed as A lines.
B ii	Lines carrying intensive traffic even though the speed is less than 60 miles per hour.
C	Lines subject to maximum speeds of 45 to 60 miles per hour.
D	Lines subject to speeds of less than 45 miles per hour.

Note : A limited mileage of lines is classified for reasons other than speed and/or traffic density ; for example, high axle-loads of locomotives or other special uses.

The permanent-way material used is graded in accordance with the classification of the line. New material is used on A and B lines, the rails being of the standard flat bottom section weighing 109 lb. per yard, or of the former standard bull-head section weighing 95 lb. per yard. On class C lines the rails are either of the standard flat-bottom section weighing 98 lb. per yard, or second-hand rails with equivalent life ; the other material is new. On class D lines the whole of the material is second-hand, the rails being of slightly lighter weight than those used in C lines.

Discussion

The Authors introduced the Paper with the aid of a series of lantern slides.

The Chairman said that both the Authors were steeped in the subject with which they had been dealing, and it must have been comparatively simple for them to produce the Paper, because they were thinking about the subject all day long and could easily put on paper exactly what they knew. To one who was perhaps not so familiar as he had been a year ago with those matters, it seemed very simple to look at the charts which had been given, and he did not agree with Mr Maycock that they were complicated ; they seemed to be extraordinarily simple, but whether they were quite so simple as they seemed would no doubt be made clear in the subsequent discussion. The subject was a most important one to British

Railways, and the Paper and discussion should be of value not only in Great Britain but overseas; there were people abroad who still looked to the mother country for a lead in such matters.

Mr J. Taylor Thompson said that the subject of the Paper had occupied the attention of railway engineers the world over for very many years. In Great Britain it had become of special interest in about the year 1935, when high-speed trains had started to run on both the East Coast and the West Coast main lines. A speed of 126 miles per hour had been achieved, and it had been obvious that very high speeds were going to be attained in normal running. The result of that was that the range of speeds increased enormously, from the slow freight train at perhaps 25 to 35 miles per hour up to approximately 100 miles per hour. The range to be catered for in curves and cant became, therefore, a matter of very great consequence. Incidentally, that showed the importance of reducing the range of speed on railways, in order to increase the capacity of the lines as well as to simplify the position from the point of view of the civil engineer, with regard both to maintenance and to the question of speed on curves.

The problem as it had appeared in 1935, and had been discussed at very many meetings at about that time, had been that of cant deficiency; that became the crux of all the discussions. The point was that it was not practicable to cant a curve for the highest speeds attainable, especially in view of the low speeds of some of the traffic, which would result in a very high weight on the low rail if the track was canted for the highest speed. The problem, therefore, was how much higher could the maximum speed be than the speed for which the cant was provided.

In 1938 he had become particularly interested in the subject, and had written a Paper which was mentioned by the present Authors. In connexion with that, he had arranged some tests, though they had not been nearly so elaborate as those described by the Authors. Mr Graff-Baker, of the underground railways, had arranged to carry out some tests on underground stock, because it had appeared that a passenger sitting on a longitudinal seat in an underground train subject to braking was subjected to much the same sensation as a passenger on a train going round a curve, having that feeling inside him which created discomfort, and comfort was really the crux of the matter; comfort was under discussion, not safety.

Mr Taylor Thompson therefore had had those tests made on the underground railways, which corresponded very much with what was said in the present Paper. In addition, he happened to know a bridge structural contractor who, as a side-line, ran a merry-go-round at a holiday resort, and he had found out from him how fast it was possible to swing people round before they felt a similar sensation. Putting those two rather elementary tests together, the track tests and the merry-go-round, Mr Taylor Thompson had arrived at a cant deficiency of 4¾ inches. He had therefore been very interested to see that the results of the much more elaborate tests made by the present Authors had finally produced, with a fair margin of safety, a cant deficiency of 3½ inches. He could easily accept that lower figure.

As soon as nationalization of the railways had taken place, the problem had arisen, of course, of providing a common standard for the railways. He had happened at the time to be Chairman of the Track Committee, to which the subject had been remitted, so that he had a particular interest in the Paper, and he had been one of the people who had taken part in the sensation tests.

It had been of great interest to him to see in the Paper for the first time —he had handed over the Track Committee to Mr Everard before the work was over—that the sensations recorded could be plotted on not too bad a straight-line curve. It was extremely difficult to record a sensation which lasted only for a very short time—1 or 1½ second—and where it was necessary to determine as between 0 and 6 what the particular sensation was, in that very small space of time. However, it should be said that the tests had provided a very good foundation indeed, and so far as he knew the Paper contained the results of the most elaborate research into the subject which had been published anywhere in the world. As Chairman of the Committee which had arranged the tests, he expressed his own special thanks to both Authors. He agreed with the Chairman that the Paper, and especially the graphs, would be found to be of use all over the world, and particularly in the Colonies and on other railways of full gauge, where no doubt the results would be used as a standard.

Mr Keith Brinsmead said that the trials were of particular interest, because they were the first in which emphasis had been put on comfort. In the past, people had always worried whether it was safe, either for the train or for the track to go at a certain speed, but the tests had proved quite definitely that the discomfort of passengers came into play first.

He thought that there had been a great deal of confusion about cant deficiency. People were inclined—he had been himself—to feel that it was eating into the margin of safety, rather like overloading a bridge; but a train could go off the line only for three reasons. It could overturn, which was a phenomenal feat; it could climb over the high rail on a curve; or the track could give way. It could climb over the high rail on a curve only when the vertical load on the outer rail was too low to resist the lateral thrust, and therefore a little cant deficiency, given good track, actually contributed to safety. That was something which he had only gradually realized.

To decide what was comfortable was an exceedingly difficult thing to do. The personal feelings of the individual came into it. All those taking part in the trials had said, in the course of conversation, " I bet so-and-so will mark one sensation higher than we do," and he had done so. It went to show that it was possible to get considerable variations in trying to decide what was and what was not a reasonable speed at which to go. In Mr Loach's part of the Paper it was very clear from *Figs 6* and *7* that sensation 5 was obtained at something more than 7 inches cant deficiency on a well-aligned curve, and the same sensation at just over 4 inches on an irregular curve.

The final reduced figures used were, Mr Brinsmead thought, very logical. They allowed for all the imperfections which must creep in and particularly for the risk on, say, class " C " and class " D " lines, where second-hand materials were used. He could well remember that during the trials, on a curve which had, he thought, the worst cant deficiency, he had not been worried about stability but had said to himself " Those poor fastenings ! " He had been able to think of what was happening to the track, but had not been at all worried about the performance of the train.

On p. 518, Mr Loach had referred to the fact that a quickly applied and released lurch did not worry people ; the sensations were marked lower than would otherwise have been the case. People might be annoyed by a sudden jolt but would not be alarmed by it if, by the time that they came to think about it, the jolt had gone. They did become annoyed, however, and that was a problem which particularly applied in the case of switch-and-crossing work. On the lines of London Transport in particular that problem existed with old tracks, designed a long time ago, with far sharper turnouts than was desirable, and, what was worse, when people were standing at one end of a coach, over a bogie.

He would quote the results of a slight modification to a junction. On the District Railway, running eastwards, just before Whitechapel there was a very sharp bend to the left, and then to the right again. The old original line which ran through to the East London Railway was no longer in use, and, owing to tunnel limitations, there was a situation where they had to get 350 trains a day on each track over " B " switches and 1 in 8 turnouts. Until 10 years ago those switches had been perfectly normal undercut " B " switches, which meant that the train suddenly hit a 1 in 32 angle, which became a serious matter not only for the passengers but also for the wear and tear of the track. The switches at that junction had required renewal approximately once every 12 months. In 1942 there had been substituted some straight-cut curved switches, still " B " switches, but made 4 feet longer, and instead of being straight they were given a radius leading round almost parallel with the running rail. The result of that reduced the angle at which the wheel actually hit the switch-rail to about 1 in 76, and 3 weeks ago the third set of switches to be put in in 10 years had been installed. It would be seen, therefore, that a lot could be done to get rid of small imperfections.

Mr H. C. Orchard observed that the Paper was of particular interest to him because he had been a member of the Track Committee at the time that the tests had been carried out in North Wales. He agreed with all that Mr Taylor Thompson had said about those tests, but had had to leave the Committee before its deliberations on the subject had been completed.

In the main he was in agreement with the conclusions reached by the Committee.

He thought Mr Loach's part of the Paper showed very clearly the difficulty of the subject, based as it was very largely on sensations rather than mathematics. He thought that if it had been only a question of mathematics, Mr Loach would have found a solution in that direction. The five factors to be established were shown on p. 505. The first, the maximum permissible cant, was largely a matter of judgement—he said " largely " because of cants greater than 6 inches in other parts of the world. The second, third, and fourth were based on personal sensations, and only the last, the steepest permissible cant gradient, would appear to be capable of being checked by physical methods.

With regard to the variable cant deficiency, as a member of the former L.N.E.R. staff Mr Orchard was familiar with its application, but he still had some doubt about whether there should be a different maximum deficiency for classes " A " and " B " lines on the one hand and classes " C " and " D " lines on the other. Apart from the complication which that involved, in the recently-published rules for speeds of trains on curves the differences in the calculated speeds were comparatively small. On 60-chain curves, for instance, where the actual cants were 1 inch, 2 inches, 3 inches, and 4 inches, the differences in the maximum permissible speeds in the higher ranges were $8\frac{1}{2}$, $7\frac{1}{4}$, $6\frac{3}{4}$, and $6\frac{1}{4}$ miles per hour respectively. It could be argued that by reason of the heavier manning and consequent higher standard of maintenance and the better material on the " A " and " B " lines they were more capable of withstanding additional side thrust, but he would suggest that renewals on those " A " and " B " lines, when they became due, were often as much attributable to the condition of timber and fastenings just as to that of the rails, and therefore they did form a weak link in the chain of the track.

A further point in that connexion was that there was at least one class " A " line on British Railways which was so tortuous in its route that it was a matter of some difficulty to introduce transitions of desirable length between reverse curves. A maximum cant deficiency of $2\frac{1}{2}$ inches for all classes of line would help in that respect where existing cants were small.

It was a pity that the minimum cant gradient had been fixed at 1 in 300 and not related to rail length. It was much easier for gangers to remember terms such as 2 inches or 1 inch in a rail length than to remember something like 1 in 25, which was a figure which was strange to them. From that point of view 1 in 360 might have been a better figure. It was appreciated that where that steepest cant gradient was called for, the curve would be monumented, but if the monuments should be displaced the gangers must be relied on to maintain the correct cants until the monuments could be replaced.

Why had it been decided to make the desirable length of a transition $1\frac{1}{2}$ times the permissible minimum length? By inference the Paper emphasized the importance of a high standard of maintenance on transition curves which, because of the site limitations, were shorter than the desirable length. That might well constitute an additional factor for assessing manpower on lengths where there were many such curves and where the formation was poor.

Mr L. G. B. Rock who remarked that he also had been a member of the Track Committee, and consequently had nothing to criticize in the Paper, observed that on p. 505 Mr Loach had given the factors which had been taken into consideration in selecting the maximum permissible cant, and referred to the case of a van having been overturned by a gale. Mr Rock had recently referred to a Paper[1] presented to the Institution which dealt with the same subject and whose Author had deduced that it was unwise to specify a maximum cant of more than 4 inches, in view of the fact that an unloaded stationary covered van of a type common at that time, 27 feet long and of 9½ tons tare weight, could be overturned by a wind force of 20 lb. per square foot. That wind force, according to the only formula which Mr Rock had been able to find, was equivalent to a gale of 63·6 miles per hour. He had been sufficiently interested to see what the effect would have been if the track had been level, and had found that it would have required a wind velocity of 68·7 miles per hour, so that the difference between the speed of the gale required to blow the wagon over on level track and on track with 4 inches of cant seemed to be very slight. In fact, with a 6-inch cant, which was recommended by the Track Committee as the maximum, the velocity to overturn the same vehicle would be 61·5 miles per hour. He submitted that the difference in those velocities was of little consequence compared with the advantage to be gained in permitting the extra 2 inches of cant, or, of course, of cant deficiency if the vehicle was moving round the curve and the wind was blowing towards the outside of the curve.

He commented on the remarkable agreement on the part of the thirteen "guinea-pigs" on each of the sensations recorded in turn. They had not all estimated the same sensation number, but the range on the whole, as Mr Loach would probably confirm, had been remarkably low. It was significant that although some of those who had taken part had a slightly more critical assessment of comfort than others, nevertheless the difference in sensation numbers on successive curves, as recorded by the various observers, was very consistent.

The fact that they considered on the average that a degree of discomfort began with 7 inches of cant deficiency had struck him on reflexion as being remarkable, in view of the fact that when going round an ordinary sharp curve on an uncanted road in a motor car, or going round a roundabout, at a speed of, say, 25 miles per hour, it was not unusual to attain a cant deficiency of 18 inches to 2 feet, and yet no particular discomfort was felt. It seemed to him that in a railway train people were influenced to some extent in their assessment of comfort by what they had come to associate with travelling by train, as opposed to travelling in a car. Whether the height of the passenger above the surface on which the vehicle ran contributed to that difference he would not like to say, but it had struck him as being an interesting feature.

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[1] J. W. Spiller, "High Speed on Railway Curves." Min. Proc. Instn Civ. Engrs, vol. CLXXVI, pp. 83–84 (Jan. 1909).

On pp. 534 and 535 Mr Maycock described the selection of the virtual transition. Mr Brinsmead had mentioned the effect on passengers standing over the bogie at the end of a coach. The point should not be overlooked that the fortunate passenger in the centre of the coach travelled over a path which was that of the virtual transition, but the passenger over the bogie was subjected to an instantaneous application or relief of radial acceleration as the bogie passed over the tangent point. In switch-and-crossing work, even with a curved-blade switch where there was still a slight angle of divergence, a passenger either in the centre of the coach or over the bogie experienced an instantaneous change, but that was of little if any more significance than the other disturbances to which Mr Loach had very rightly drawn attention—the impact of wheel flanges against check rails, passing over crossing gaps and so on. For that reason Mr Rock supported the contention that it was wise to exercise a slightly tighter control over cant deficiency in switches and crossings, than in plain track on classes "A" and "B" lines.

Lieut.-Col. G. R. S. Wilson thought that the Paper and all that it dealt with represented true progress. Instead of the varying and perhaps unco-ordinated methods of the past a thorough study of the problem had been made, with the experience of all the former Companies behind it; and the results had now been codified in the rules and instructions which, he understood, were being issued by the Railway Executive. Not only had very careful mathematical theory been used, but also practical considerations had been taken into account, including the human factor. The agreement on sensations, as plotted on the graphs, seemed to be extraordinary, considering the different reactions of different people.

Colonel Wilson remembered the work which had been done on the problems presented by the high-speed trains which had been introduced in 1935 on the East Coast main route. He had travelled on one of the first test runs, and he remembered that right up to the outbreak of war, no doubt because of the attention and research given to the track, there had been a steady improvement in the running of those high-speed trains. The maximum speed permitted in actual service had been, he believed, 90 miles per hour, but he thought that they used to go quite a lot faster sometimes.

He remembered going round the Hatfield curves often enough in his journeys to the North. The speed used to rise to approximately 90 miles an hour with those light high-speed trains, and he would have described the sensation experienced as a certain feeling of insecurity, but he used to console himself with the thought that the outside wheels were thereby being pressed more firmly on the rail all the time.

On looking through some of the Ministry's reports on main-line derailments, he had noticed that at Hatfield there was 2¾ inches of cant on a 76-chain curve. On the other hand, at Polesworth on the L.M.S. Trent Valley line, on a 76-chain curve there was 4¼ inches of cant. Both were high-speed routes. Presumably that represented the difference in practice which prevailed among the former Companies.

The question of maintenance was necessarily dealt with lightly in the Paper, by reference to "normal deterioration," "imperfections," and "discrepancies," but he would like to relate that to a point which Mr Brinsmead had made, or perhaps it would be more correct to say to a point which Mr Brinsmead had not made. Mr Brinsmead had said that one cause of derailment was overturning. It was well known that there had been few cases of overturning in the past; Colonel Wilson could recollect only three, namely, those at Salisbury, Shrewsbury, and possibly Canal Junction, Carlisle, in 1930. Another cause which had been mentioned was the climbing of the flange over the high outer rail of the curve. In several cases which it had been the duty of Inspecting Officers to investigate, derailment had occurred to the inside of the curve. There was nothing extraordinary about that. There were the dynamic effects of rolling and lurching, together with the side-to-side movements of a long-wheelbase engine, and the consequent relief of weight on one of the wheels on the inside. That seemed to him to be very much related to cant deficiency, and he would also suggest that there was a relationship between cant deficiency and standard of maintenance.

A cant deficiency of 3½ inches for "A" and "B" lines, and of 2½ inches for "C" and "D" lines, was the maximum permissible. The greater the cant deficiency, the more important was the maintenance of the regularity of the cant, because in any case, during steady running on a curve with a cant deficiency, it was axiomatic that there was some degree of relief of the designed weight on the inside locomotive wheels. That was inherent, and if it were increased by lurching and rolling, and by dynamic effects and irregularity of cant, the higher the designed deficiency at any given speed the greater was the proportional relief of weight on the inside of the curve.

Track maintenance was not susceptible to the same standards of accuracy as mechanical engineering, since natural materials were involved. As a supplement or sequel to the present Paper, might it not be very useful to draw up something in the way of tolerable standards, by classes of line and types of curve, for irregularity or permissible variation from perfection? He suggested that such standards would no doubt be very difficult to get down to because they were not so susceptible to scientific treatment as present design standards, but how far was it possible to go with safety? What could a permanent way inspector tolerate? What could a ganger tolerate? What could be tolerated before it was necessary to restrict the speed?

He had been looking up what the maximum cant gradients in a few cases of derailment had been, and what was the actual worst cant gradient found. (He was not now referring to the Weedon accident, with which he had been concerned recently, because that had not been a track case at all but had had an entirely different cause.) He had found that the cant gradients ranged from 1 in 300 at the worst to 1 in 1,080. There were, however, reversals of cant in some cases, high-low-high-low in wave form, which had set up rolling and lurching and had led to derailment to the inside of the curve. He hoped that consideration would be given to a sequel to the present very valuable Paper on the lines he had suggested.

Mr A. I. Emerson commented on the practical application of the code of practice which the Authors had suggested. His remarks were based upon experience with a similar code of practice which had been introduced on the L.N.E.R. a few years prior to nationalization.

From the point of view of the designer the code of practice given in the Paper was very helpful, because it almost reduced the determination of maximum permissible speeds on curves to a simple arithmetical calculation admitting of one solution only, and even that work had been almost eliminated by means of the graphs which Mr Maycock had shown. Mr Emerson said "almost" because the code did not directly indicate the amount of cant which should be used in the calculation; the designer was left with the task of assessing the actual amount of the cant after giving consideration to the particulars which were set out on p. 530, namely the speed limit, the speed restrictions, the gradients, and the nature of the traffic. It was therefore essential that those factors should be fully considered, and, as Mr Maycock had pointed out on p. 540, that was best done by making a review of conditions on the route, as opposed to individual consideration being given to each curve.

Mr Emerson stated that a speed-distance chart was an essential preliminary to making any such review and showed a slide of a chart having three speed lines for each track:

1. The speed limit and speed restriction line;
2. The speed of an ordinary express train (obtained during a Hallade record run);
3. The record for a high-speed train (representing the maximum requirements of the Operating Department in the near future).

The other information given on that simple form of speed/distance chart had been limited to the magnitude and direction of curvature, gradients, and mileages. The designer might, however, require a chart to a larger scale, so that further details of curvature could be added, together with a line diagram of the permanent way layout. If it were to be used by those responsible for the timing of trains, dimensions would be added relating the speed limit and speed restriction line to the timing points, such as signal boxes.

The code of practice could be applied to curves whether they had been re-aligned or not. In the latter case the designer had to carry out additional work before the factors governing cant could be properly assessed and the code of practice applied. Mr Loach had distinguished between curves which had been re-aligned by the Hallade method and those which had not.

On routes which had been re-aligned, a suitable speed/distance diagram might already be available; but, owing to the work having been spread over some time and carried out by different individuals, it was possible that uniform treatment might not have been given to the very important governing cant. Overall consideration of those factors for the route, and especially of the estimated speed limit, would enable the code of practice to be applied to determine the maximum permissible speed on each curve on a consistent basis.

On routes which had not been re-aligned, a reliable speed/distance chart would not be available, and before the code of practice could be applied the designer had to carry out additional work on the ground in taking particulars of the curves. He had then to regularize the existing versines and cants which he had obtained and to use his judgement in distinguishing between irregularities due to faulty maintenance and those which could be put right only by major or minor re-alignment schemes. The extent to which re-alignment was required to avoid an excessive number of speed restrictions could be decided only after considering the route as a whole; again, if a speed/distance chart were prepared, the work was greatly facilitated, and the code of practice could be properly applied even to curves which had not been previously re-aligned.

Engineers responsible for fixing maximum permissible speeds would be very interested in the tests which the Track Committee had carried out in order to assist them in determining the limiting values for deficiency of cant, and for the ratio of change of cant and of deficiency and so forth. Mr Emerson showed some slides illustrating actual running at higher speeds (about 60 miles per hour) than those for which the Track Committee had recorded their sensations (40–45 miles per hour). The curves were 32-chain reverse curves on an important main line carrying express trains.

One slide showed the versines and the cants which had actually been applied to the track in accordance with the code of practice formerly used by the L.N.E.R. The maximum permissible speed according to that code of practice was 60 miles per hour, and the limiting factors were the rate of change of cant deficiency and the rate of change of cant, both of which were at their limiting value of 1½ inch per second. The slide also showed the cant which would be applied according to the code of practice suggested by the Authors. The maximum permissible speed had been increased to 65 miles per hour, and the limiting factor was the deficiency of cant on the circular part of the curve, which was then 2 inches. The rate of change of cant was slightly below its limiting value at 65 miles per hour; it was only 2⅛ inches instead of 2¼ inches per second. The versine-cant diagram displayed by Mr Emerson illustrated the differences between the Authors' code and the former L.N.E.R. code, which had been rather more conservative.

Mr Emerson showed a reproduction of an actual Hallade record at an average speed of about 68 miles per hour on the same curves. The deficiency was ¾ inch (or 40 per cent) more than that recommended by the Authors, and on the transition the rate of change of cant was 2¼ inches per second, which was the Authors' limiting value. In spite of that and of the high speed, the Hallade record showed the riding to be reasonably good both on the circular part of the curve and on the transition. However, a regular traveller travelling at 68 miles per hour would probably experience sensations of a somewhat different nature from those felt by the Track Committee during their tests, which had been made at a rather lower speed. Sensations were difficult to describe, but it might perhaps be said that the traveller would " anticipate " discomfort. When crossing work had been reached further round the curve rather bad riding had occurred, and was probably due to the lateral forces caused by the check rails.

Mr Emerson appreciated that that one example did not do much to support the conclusions arrived at by the Authors for fixing the limiting values for cant deficiency and so on, but he thought that many more cases could be shown which would help to justify the code of practice which they had suggested. It would perhaps be interesting to see whether on flatter curves with a lower cant a higher deficiency could be permitted, because, as Mr Taylor Thompson had said, before the war, high-speed trains had been running on flat curves (which had been, of course, very well maintained) at much higher deficiences and at speeds of over 100 miles per hour.

Mr W. E. Gelson observed that the provision of sufficient cant to meet the requirements of comfort was not always enough to ensure a satisfactory safety margin at high speeds, particularly when the driver had somewhat under-estimated the speed. That had been appreciated by Mr Loach, as was shown by the remarks on pp. 514 and 515. That aspect required reference to experiments on the riding of locomotives, a large number of which had been made during the past 20 years in various countries. Those experiments had shown that a given value of cant deficiency at speeds below 45 miles per hour, on broad and standard gauge, had a far less disturbing effect on the track than it had at the higher speeds. It would therefore appear that the amount of cant deficiency provided should be closely related to normal maximum speed.

There had in the past been more derailments on curves due to lateral thrust than to overturning under the action of centrifugal force. On p. 531 Mr Maycock had stated that it was not always necessary or desirable to use the maximum allowable cant deficiency, in order to facilitate track maintenance. That was extremely sound policy. There were doubtless not a few main line curves where greater speeds would be practicable if a cant greater than 6 inches could be provided. In cases where the speed of the slowest train was sufficiently high, Mr Gelson suggested that some increase above 6 inches could be given, provided tapered sleepers were used, possibly in conjunction with a special sub-frame, in order to ensure stability of the under-ballast.

Mr K. H. Tredinnick said that on p. 537 Mr Maycock had referred to the speed of trains through turnouts, and on p. 536 had given a formula for relating the speed at the tangent point of a curve where there was no transition. Mr Maycock had made some mention of his reason for the higher speeds allowed through turnouts, but Mr Tredinnick was not clear about that and would like some amplification. He thought that if the speed round a curve was restricted on account of lack of transition in plain line, there should be a greater restriction provided through a turnout, where there were more difficulties, particularly in divergence of switch.

Mr Loach had referred on p. 519 of the Paper to the *P*/*W* ratio. Mr Tredinnick was not clear whether the minimum cant gradient of 1 in 300 which was referred to had been arrived at as a result of considering the twisting force due to the actual cant run-out on which Mr Loach superimposed a 1 in 300 theoretical cant deficiency run-out. It was thought that the *P*/*W* ratio due to the twist of the actual cant run-out would increase considerably when coupled with an unequal wheel loading due to cant deficiency.

Mr Rock, Mr Loach, and Mr Taylor Thompson had said that there was very little harm in cant deficiency, and that the experience of a person travelling in a train would be one of no discomfort even up to a figure of possibly 4 inches on a fairly badly-aligned curve. If that were the case, and if the recommendation of 3½ inches mentioned by Mr Loach were based only on stress considerations, particularly on fastenings, Mr Tredinnick would disagree with the Table and with the recommendations made by the Authors that cant deficiency should vary with actual cant, which was referred to in Table 5, p. 527. He thought that in considering cant deficiency it was useful to get back to first principles on the subject.

Cant deficiency was no more than a convenient name, in the absence of a better one, for an unbalanced centrifugal force which came into effect when travelling at a speed greater than the equilibrium speed. On any curve it was also possible to get an inwards force due to cant surplus; in other words, an inwards force due to a slow-moving train. He thought that in the interests of track economy, and particularly in respect of fair wear and tear of track, it would be better to balance the outwards force due to the speed above the equilibrium speed with that of the inwards force due to the difference between the equilibrium speed and the speed of the slower train. The condition to allow those two forces of cant deficiency and cant surplus to be equal would be

$$V_m^2 - V_e^2 = V_e^2 - V_s^2,$$

where V_s denoted the speed of the slower train, V_m the maximum speed in accordance with Mr Emerson's timing-of-trains diagram, and V_e the equilibrium speed. That could be developed further to give the following formulae :

$$E \text{ (actual cant)} = \frac{0{\cdot}03}{R} (V_m^2 + V_s^2)$$

and

$$D \text{ (deficiency of cant)} = \frac{0{\cdot}03}{R} (V_m^2 - V_s^2).$$

Those were reasonably simple formulae and gave excellent practical results. Speed restrictions could be defined by the maximum of $E = 6$ inches or $D = 3\frac{1}{2}$ inches, whichever was reached first.

The Authors had dealt at great length with various theoretical considerations in arriving at a $3\frac{1}{2}$ inches cant deficiency and so on, and stated that they had also experienced all sorts of sensations on the Bettws-y-Coed line; then on p. 526 of the Paper they suddenly adopted an empirical formula which appeared to be without proof, and which made cant deficiency a function of actual cant. So far as Mr Tredinnick was able to interpret their Paper, their excuse for that was that maintenance problems demanded a varying cant deficiency, and they inferred that those maintenance considerations applied only to the fastest trains. On p. 531 Mr Maycock had, in point of fact, stated that on lines where freight traffic was considerable a maximum speed restriction could be imposed on the faster trains. That in itself appeared to be a confession that the effects on track from the point of view of maintenance were as much attributable to the slow-moving traffic as to the fastest traffic. Mr Tredinnick was of opinion that a slow-moving train on a heavily canted road probably had more ill-effects than a fast train, as it bumped and bored its way along contributing to gauge-widening and many other evils, all of which defects reacted on the comfortable running of the fast trains. In his opinion, therefore, it would have been better either to have adopted a formula such as he had indicated, which catered to some degree for the ill-effects of both the slow and the fast train, and was economical from a rail wear and maintenance point of view, or to have confined the recommendations to those contained in Table 4, and to have left the rest to the engineers on the site, who knew all about local site conditions and the ranges of traffic to be dealt with, and who could be trusted to get the maximum out of the job from the point of view of track strength, track life, speed, and good time-keeping.

Table 5 allowed for reduced deficiency at the higher actual cants, and as such would not fully cater for the slower train, but tended to have the effect of causing increased maintenance problems and track wear, in addition to being a restrictive practice which would tend to reduce speeds.

With regard to the "C" and "D" lines, those were covered in any case by an overall speed restriction, and it was usual to find on that class of line a greater proportion of slow-moving traffic. He thought that the fact that the Authors had further restricted permissible cant deficiency at the higher ranges of cant in that case would worsen the situation and increase wear and maintenance problems due to slow trains. He considered that it was a pity that a stop had not been made at the recommendations in Table 4, leaving all questions going beyond that to the engineers responsible for the work, who could choose the actual cant and cant deficiency within the maximum limits laid down to suit local conditions.

⁂ **Mr G. D. S. Alley** remarked that Mr Loach would be interested to know that comfort was considered by the L. & N.W. Railway when deciding on the allowable deficiency of cant. It had been found that to travel at 20 miles per hour round a curve of 10 chains radius without any superelevation did not produce any undue feeling of discomfort, and was perfectly safe. The figure of 2⅝ inches was therefore used as a maximum allowable deficiency of cant on that Railway, and for some years after the formation of the L.M.S. on quite a large area of the latter.

With regard to Mr Loach's comments on the difficulty of maintaining track with more than 4 or 4½ inches of cant, Mr Alley differed and stated that surely that depended solely on the traffic and the speed of such traffic on the track in question. For example, with a 50-chain-radius curve and practically all the traffic passing at speeds between 65 and 75 miles per hour, there should be less difficulty in maintaining good top and line with 6 inches of cant than there would be in maintaining straight track–formation, ballast conditions, etc., being comparable. On the other hand, should there be a large amount of traffic at speeds of 40 to 50 miles per hour with only one or two trains at speeds of 60 to 70 miles per hour, it would be difficult to maintain a cant of 6 inches, and such a cant should not be used in that case.

Mr Alley noted from *Fig. 16* that three half-chord points were shown on the transition together with the half-chord points at each end. On *Fig 17*, however, only two half-chord points on the transition were shown, and it was considered that the necessity of a minimum of five half-chord points per transition, including those at each end, should be stressed to ensure accurate setting-out and ease of maintenance.

A minimum length of transition was again quoted on p. 535, whilst a desirable transition of only 89 feet was shown on *Fig. 17*. That, presumably, was to justify a cant deficiency, but surely each case required consideration on its merits within the limits laid down. If on a compound curve it was considered good practice to provide the cant for equilibrium speed on one part, why suggest to a designer that he should provide less than that required for equilibrium speed elsewhere on the same curve, unless the site conditions were such as to make that necessary or the cant required for such speed was more than the limit of 6 inches laid down previously?

Mr D. H. Coombs observed that there had apparently for some time been fair agreement in Europe that the maximum value of cant, expressed as a tangent in the form $\frac{\text{cant}}{\text{gauge}}$, should be between 0·08 (German railways)

⁂ This and the following contributions were submitted in writing.—SEC. I.C.E.

and 0·1066 (French railways), with exceptionally the P.L.M. system using 0·131. Most European railways appeared to favour a maximum cant equivalent to $\frac{1}{10}$ gauge, where gauge was the gauge between rail axes.

On 4-foot-8½-inch + 2¾-inch gauge, 6 inches was equivalent to $\tan^{-1}$ 0·101. The decision for 6-inches maximum cant could therefore reasonably be regarded as in line with general European practice.

There seemed to be a fair measure of opinion on the Continent that 3½ inches of cant deficiency, or about 1·9 feet per second per second unbalanced centripetal acceleration, was reasonable, but that deficiencies greatly in excess of that might be dangerous, though the Italian railways used up to 3·285 feet per second per second, or about 6 inches of cant deficiency. From the passenger tolerance point of view the experiments appeared to show that 4 feet per second per second (which was equivalent to 7¼ inches cant deficiency) was a reasonable imposition as such, but that owing to line imperfections it was wiser to limit it to rather less than half of that value. The decision for 3½ inches of maximum cant deficiency could therefore be described as orthodox.

Referring to Figs 2, Plate 1, the portion of the 40-miles-per-hour Askania trace relating to the entry transition of Curve No. 5 showed a maximum unbalanced radial acceleration of about 3½ feet per second per second. But considering the first differential of that trace, that was, the rate of increase of unbalanced radial acceleration, it appeared that in the transition to curve No. 5 a rate of increase of about 6 feet per second per second in a second (equivalent to about 11 inches gain of cant deficiency in a second) was registered during the actual achievement of about 3½ feet per second per second. Here it would clearly be the $Pt = mft$ aspect which would give discomfort rather than the $P = mf$ evaluation. Furthermore, since the total journey-time through that transition was about 2 seconds, it would, Mr Coombs thought, be quite impossible for the observers to distinguish the impulsive aspect from the growth of cant deficiency as such. The same sort of complication masked subjective evaluation of the constant unbalanced radial acceleration over circular curve No. 5, and to a somewhat lesser extent on circular curves Nos 3 and 4.

It followed that before cant deficiency itself could be measured subjectively, a very long curve in a very high state of curve regularity was necessary, otherwise the observers could not hope to distinguish between the effects of cant deficiency and those of line deficiency.

From what had been said previously it would appear that the possibility of subjectively measuring a quantity compounded of tilting, a gradual increase of cant deficiency, and irregularities of line, was not very great unless the transition was long and regular, with an easy cant gradient. The failure of correlation of the sensation numbers with measured cant deficiencies in the second range of transition curves was not surprising. The wonder was that the array of sensation numbers relating to curves Nos 1 to 14 was as statistically as significant as it was ; though if the gains and losses had not been averaged, as he assumed they had been, the scatter would have been more random even with the Nos 1 to 14 series.

The maximum rate of loss or gain of 2¼ inches per second adopted therefore appeared to be slightly conservative in the light of the relevant observations, but was probably an adequate ceiling for most track designers.

The publication of the random scatters obtained on curves Nos 15–52 was, he thought, to be appreciated as a gesture of scientific honesty.

He remembered having, in the past, put cant on to a sharp curve at what he had then thought to be the maximum safe rate of 1¼ inch in 33 feet, or 1 inch in 26·5 feet. From his knowledge of the train speeds, as electrically recorded, he had known that the higher speed was 34 miles per hour, thus involving a gain of cant of nearly 2 inches per second. At that time he had regarded that as somewhat daring, because Mr Taylor Thompson, in his Paper[1] published in 1938, had suggested that whilst American Railroads adopted 1·22 to 1·5 inches per second, his own experiments had shown that 1·7 inches was quite comfortable; though he counselled that only that 1·5 inch could be adopted with safety. However, travelling over the 1 in 320 gradient at 34 miles per hour Mr Coombs had found that he could just feel the approximately 2-inches-per-second roll and that, he thought, was because he was conscious of its location. It was a piece of track that he had subsequently travelled over frequently, but without thinking about it at the right moment he never noticed it.

He was in doubt as to the expediency of using so steep a cant gradient as 1 in 300 at one sort of location, a sharp reverse curve without any intervening straight, not with regard to wagons but with regard to locomotives with more than two coupled wheels, and with travelling cranes with two or more fixed axles. With those vehicles there might be a danger that the rigid wheelbase and very stiff springing, in combination with conterminous 1 in 300 gradients, and defects in track maintenance, might cause a shifting of weight, not from the inner wheels to the outer wheels, but from the inner axles to the outer axles when, if for one reason or another, the springs could not easily follow the contour of the rails; then there was a possibility of derailment following an acceleration or deceleration surge in the engine or train.

That position could be made worse by the presence of a vertical curve which was concave upwards across the transitions. It followed that the definition of a radius minimum for vertical curves was a desirable complement to the definition of a maximum cant gradient.

Mr A. W. T. Daniel observed that the proposed new limits for cant, cant gradient, and rate of gain of cant appeared to be logically developed from basic considerations and experimental observation, and were a great advance on the old-fashioned rule-of-thumb methods; but certain questions arose regarding cant deficiency. On p. 526 of the Paper formulae were given connecting D, the cant deficiency, with E, the actual cant, which had the effect of giving D decreasing values for increasing values of E; the reason advanced for that, on p. 516, was the greater influence of track imperfections at the high speeds associated with the higher cants. But since it would seem more likely that the highest speeds would be developed on straights or slightly curved track, where the cant would be either zero or small, an equally valid case could be made out for increasing D as E increased, instead of the reverse.

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[1] "Railway Track Work for High Speeds." Paper read before Yorks Assocn Instn Civ. Engrs, March 1938. Abs. printed in J. Instn Civ. Engrs, vol. 10, p. 405 (Dec. 1938). (The full MS. and illustrations may be seen in the Institution Library.)

A more logical solution would apparently be to decide on a constant value of D for all values of E on plain curves; or else to adopt a method as suggested by Mr Tredinnick, which led to the equations:

$$e = \frac{s}{2gr}(v_m^2 + v_s^2) \quad \text{and} \quad d = \frac{s}{2gr}(v_m^2 - v_s^2)$$

where v_m and v_s denoted the greatest and least speeds at which trains were expected to traverse the curve, all in foot-second units. Actually, since a train might be stopped on a curve, in the limit $v_s = 0$ whence $d = e$, which showed the impossibility of allowing for all conditions, and it would probably be simpler, and equally logical, to adopt a constant value of D for plain curves as he has just suggested.

Turnouts presented a special problem, since at the switch points there was a sudden change of direction, usually associated with zero cant. According to pp. 526 and 537 of the Paper that was to be dealt with by using the formula as for classes "C" and "D" lines, and taking R as the radius of the turnout curve. But the governing factor was the sideways impulse set up at the switch points, which depended on the mass and velocity of the vehicle and the angle of the switch rail, and was independent of the subsequent switch and turnout radii. That could not be dealt with by canting the track in the usual manner, and to assume a radius equal to that of the turnout, even when it was less than the switch radius, was at best a sweeping approximation. A more logical procedure would appear to be to lay down maximum speeds for each type of switch rail, which would have to be determined by experiments conducted from the point of view of passenger comfort.

The high degree of uncertainty governing the speeds at which turnouts could be entered was illustrated by the manner in which negative cant was dealt with. Taking the example on p. 537, if $E = -1$ inch, then D should be $2\frac{3}{4}$ inches according to the formula, but the value given of $2\frac{1}{4}$ inches could be obtained only by ignoring the sign of E and using the formula as if it were positive. The reasons for that procedure were not quite clear, and again it would appear to be equally logical to adopt constant values of D as he had suggested.

A matter which did not seem to be mentioned in the Paper was that of compensating curves. To suggest an example, considering a line having a ruling gradient of 1 in 200, if a curve had to be laid to that gradient and the limiting value of 1 in 300 for cant gradient were applied to the outer rail, then that rail would have an actual gradient of 1 in 120, and the centre of gravity of the train would have to be lifted through the average gradient of 1 in 150. The Authors of the Paper had stated that each individual case should be considered on its merits, but it would be of interest to know if it was intended in the future to draw up a set of rules governing problems such as that just considered.

Mr Loach, in reply, expressed his thanks for the kind reception given to the Paper, and thanked all those who had made constructive comments and criticisms.

Several speakers had referred to the question of the different sensations, and to the possibility that the thirteen observers who had submitted themselves to the tests had recorded different sensation numbers. They had done so to a slight extent, but on the whole the agreement had been good. Two members had consistently recorded higher than the average, and two consistently lower, but eight or nine of the thirteen had usually recorded within 20 per cent of the average figure, taking the totals for each test. Even though there had been little difference in the sensations described by two consecutive numbers, some of the observers had used half-numbers on occasions. It might be of interest to mention that No. 6 had not often been used, except by one member ; even on the high-speed run most of the observers had recorded 4's and 5's, and so six numbers appeared to cover the required range of sensations quite satisfactorily.

Lieut.-Col. Wilson had spoken about flange-climbing derailments. It was true that most derailments occurred as a result of flanges climbing rails rather than from vehicles overturning, but Mr Loach thought the flange would only climb up the inside rail of the curve if the riding were very unsteady. Normally, the deficiency of cant would help to force the leading flange of the vehicle even harder against the outside rail than would be the case if the vehicle were running at equilibrium speed. At the same time, of course, more vertical weight was carried by the outer rail, that weight having been taken off the inner rail.

The suggestion that permissible limits of irregularity should be determined was very interesting. Most of the limitations mentioned in the Paper had been imposed as the result of sensations, so that the question of safety would arise only with considerably greater rates of gain or loss of cant and of cant deficiency than those which had been suggested in the Paper as limiting values to be used in design. The question of cant gradient was a different one, and in that respect he was about to embark on some work on vehicles other than four-wheeled wagons to find out more about it. He hoped that he would still find there was an adequate margin of safety on a 1-in-300 gradient, and to establish that he was proposing to make tests on artificial cant gradients steeper than 1 in 300.

Mr Tredinnick had raised the question of the P/W ratio for long four-wheeled vehicles. The vehicles had been weighed, when placed on artificial cant gradients on a weighbridge, to find the redistribution of weights when unladen, and hence the *P/W* figures had been calculated. Admittedly, if there was very unequal loading, such as a diagonally placed load which radically redistributed the weights, the situation might be worse than the figures had suggested, but a load should not be permitted to run unless it was within reasonable limits of symmetry, and then it would not, Mr Loach suggested, alter the distribution of weights to such an extent as to make the position worse than when the vehicle was unladen.

Mr Tredinnick also seemed to disagree with the idea of grading down the cant deficiencies. Mr Loach suggested that it was largely a question of maintenance. He had mentioned in the Paper what the Track Committee felt about the question of maintaining track which was steeply canted; it was usually more difficult than with track which was not so steeply canted. If there were large cant deficiencies in addition, it was going to be a real problem. The whole situation had to be considered in the light of what could be done with a reasonable amount of man-power and without having to give undue attention to any particular portion of the line.

Mr Alley's description of how the maximum permissible cant deficiency had been settled on the former L. & N.W. Railway was interesting, but, apparently, very incomplete. Mr Loach suggested that if that figure had been settled so long ago, the Track Committee were fully justified in making trials with modern recording instruments. The result had given an increase of more than 50 per cent in the permitted maximum deficiency, and the increase was a boon at places where no cant could be allowed.

Mr Loach agreed with Mr Alley that a curve with 6 inches of cant would be no more difficult to maintain than any other if all trains were to run round it at equilibrium speed, but, for a specified speed, the more highly canted curves were of smaller radius and any deviation from the equilibrium speed gave a greater deficiency or excess of cant than would be the case on a large-radius curve when the same deviation from equilibrium speed occurred. Mr Loach suggested that, in practice, there were relatively few curves with 6 inches of cant over which all trains ran at speeds within 7 per cent of the equilibrium speed. (That 7 per cent was based upon Mr Alley's figure of 65 to 75 miles per hour for a curve for which the equilibrium speed was 70 miles per hour.)

Mr Coombs has referred to the Askania accelerometer records; those had, of course, included the rolling of the coach, which accounted for many of the small irregularities in the traces obtained. To cause such "waves" in the trace little rolling of the coach was necessary—so little, in fact, that it would be barely perceptible to the passenger. For assessing the results they had been ignored, measurements having been obtained from the best straight line drawn through each portion of the trace. Mr Loach considered that that course was quite justified because a passenger did not experience the same discomfort owing to rolling (which caused variations from a mean line of the order of those under discussion) as that which he experienced owing to unbalanced centrifugal force. In much the same way a passenger barely sensed 2 inches excess of cant on a left-hand curve but he would be quite aware of 2 inches deficiency of cant on a right-hand curve, although both conditions would give exactly the same record on an instrument.

Mr Coombs had doubted the advisability of using a 1-in-300 cant gradient at reverse curves without any intervening straight, but Mr Loach considered that so far as twist of the track was concerned the conditions at the point of reverse were the same as elsewhere on the cant gradients. Vertical curves and defects in track maintenance were undesirable anywhere on steep cant gradients but no more so at the point of reverse than elsewhere. In fact, at speeds other than the equilibrium speed, the weight distribution at the point of reverse would be more nearly correct than elsewhere and the flanges would not be bearing hard against either rail. It should also be borne in mind that short lengths of straight between curves of opposite flexure often caused vehicles to roll more than if the curves joined.

Mr Maycock, who also replied, noted that Mr Orchard had expressed doubt about the need for two different ranges of cant deficiency, one for Class " A " and " B " lines and another for Class " C " and " D " lines, and had pointed out that the margin between the speeds allowed by the two rules was very small. Mr Maycock admitted that the difference of speed was not great, but considered that it was worth making a distinction in speed between the two groups of lines because of the slightly weaker track material used in " C " and " D " lines and the generally lower standard of maintenance.

Mr Orchard had suggested that the maximum cant gradient should be 1 in 360 so as to be a multiple of the rail length, but Mr Maycock said he would prefer that the lower figure of 1 in 300 should be retained. It should be remembered that that gradient was laid down as a maximum and that easier gradients would generally be used. He saw no great advantage in relating the gradient to the rail length, especially as rail lengths were not invariably uniform. The figure of 1 in 300 had been introduced as a result of tests made on the derailing tendency of long-wheel-based four-wheeled wagons, as explained by Mr Loach. Mr Maycock hoped that action would be taken to eliminate the worst of those wagons and that future designs of long four-wheeled wagons would be modified so as to reduce their tendency to derail on steep cant gradients.

Lieut.-Col. Wilson had mentioned the wide difference of cant on two curves of the same radius, both being on high-speed lines. Mr Maycock considered that the application of the rules given in the Paper would tend to reduce such differences of practice, but, as he had pointed out, it was impossible to give a precise rule for the amount of cant to be provided. It was necessary to decide on the speed for which equilibrium cant was to be provided, and that was a matter for judgement, taking into consideration the circumstances in each case.

Mr Maycock attached importance to route-speed diagrams of the type shown by Mr Emerson. They gave a clear picture of the speed conditions over the whole route and were of great assistance when a series of curve adjustments were being dealt with. Since they showed actual recorded speeds, as well as maximum permissible speeds, they were sometimes useful in proving that higher speed limits were unnecessary, in order to shorten train timings, if full advantage were to be taken of the present maximum permitted speeds.

The curve realignment diagrams shown by Mr Emerson were based on principles used on the former L. & N.E. Railway, which were very similar to those presented in the Paper. The Hallade records taken on those curves showed that the application of those principles would give comfortable running.

Mr Maycock expressed interest in the formulae quoted by Mr Tredinnick and would like to study their effect.

In reply to Mr Alley's comments, Mr Maycock observed that the actual number of half-chord points shown in *Figs 16* to *18* had no precise significance; the number would vary according to the length of chord chosen and the length of the transition. He did not consider it necessary to adhere to a minimum of five such points where short transitions were used. The example illustrated in *Fig. 17* (p. 533) was intended to show how suitable values of cant, deficiency of cant, and length of transition might be obtained from the graphs, and how the cant and the deficiency of cant should change from one part of the curve to the other. Clearly, each case had to be considered on its merits and a suitable selection of values made. He did not consider it necessary in all cases to provide equilibrium cant on each part of a compound curve, nor was it always desirable to do so. If that had been done in the case illustrated a longer transition would have been required.

In regard to Mr Daniel's criticism of the graded cant deficiency, Mr Maycock pointed out that the greatest need for a high cant deficiency arose on curves where no cant, or very limited cant, could be provided. That fact, together with the maintenance considerations mentioned by Mr Loach, justified, in the Author's view, the graded-down maximum cant deficiency as compared with the uniform deficiency suggested by Mr Daniel.

The formulae quoted by Mr Daniel were another form of those suggested by Mr Tredinnick. By their use, cant was selected to suit an intermediate speed in such a way that the outward force due to cant deficiency at the highest speed equalled the inward force due to surplus cant at the lowest speed. Presumably it was claimed that by thus equalizing those opposite forces wear and tear of the track was reduced to the minimum. That might be so in particular cases, but since the formula:

$$V_m^2 - V_e^2 = V_e^2 - V_s^2$$

took no account of the weight or number of trains travelling at the two extreme speeds, V_m and V_s, or at any intermediate speeds, it was by no means certain that the total outward forces would balance the total inward forces, and result in the minimum wear and tear of the track. On p. 530 Mr Maycock had pointed out the weakness of such formulae and took the view that the speed for which equilibrium cant should be provided should be selected by careful judgement of the circumstances of each particular case.

Mr Daniel had suggested that the maximum speed for each type of switch should be determined by experiment. The severity of the divergence at the switch was, however, affected by the curvature of the main line through the turnout, and a large number of tests would be required to cover every possible variation of the angle of divergence for each type of switch. The thrust at the switch was instantaneous and was so quickly followed by that on the turnout curve that it would probably be difficult to distinguish between the two sensations of discomfort. It was unlikely that a simple rule could be formulated as a result of the tests. The Track Committee had approached the problem with a view to devising the simplest possible rule. The curvature of the branch or turnout curve in standard leads from a straight main line bore a definite relation to the angle of divergence at the switch. Both were affected if the main line through the turnout was curved, but a relation still existed between the two. It was considered that the radius of the turnout curve gave the most convenient measure of the severity of the divergence at the switch in all circumstances, whether the main line was straight or curved, and it had therefore been decided (as explained on p. 537) to base the maximum speed for the whole turnout, including the switch, on the radius of the turnout curve, and at the same time to make some allowance for the sudden thrust at the switch by using a lower maximum deficiency of cant. The simple practical rule thus obtained gave maximum speeds which experience had shown to be within the limits of comfortable running.

With regard to Mr Daniel's comments on turnouts with negative cant, the value to be taken for D (as stated on p. 537) was the same for negative as for positive cant; thus in the example given, where the negative cant was 1 inch, D would be $2\frac{1}{4}$ inches. That procedure gave the desired effect of reducing the maximum permissible speed as the negative cant increased.

Correspondence

Mr T. D. Turner observed that his particular interest in the Paper lay in the application of the Authors' recommendations to the design of curves for high-speed traffic and their effect on switch-and-crossing work.

It would be appreciated that for comfortable high-speed running and ease of maintenance, the curvature and cant through a curve should be as regular as possible so as to avoid changes in cant deficiency and excess at speeds other than the equilibrium speed.

For junction work, it was often necessary to flatten the curvature in order to reduce the cant requirement at switches and obtuse crossings and, in addition, to run at a cant deficiency on the main line through the junction. Such compounding of the curvatures could be effectively transitioned, and a reasonable cant and cant-deficiency gradient could usually be obtained, particularly if the main line carried a speed restriction through the junction.

In the case of trailing crossovers and trailing connexions to sidings, etc., it was not normal practice to compound the main-line curve or reduce the main-line cant in order to restrict the negative cant over the connexion. The application of main-line cants greater than 2½ inches through such connexions had been appreciated by Mr Maycock in the paragraph headed "Turnouts with Negative Cant," but the restriction of the negative cant to 3 inches in the switches would involve many additional main-line speed restrictions.

On such connexions, too, it was normal practice to reduce the negative cant progressively between the switches and the crossing by adzing, packing, or, in recent years, by the use of thick-base chairs. Cant gradients as high as 1 in 200 had been applied in those cases without detriment and with a considerable easement in the maintenance. Tests carried out some years ago upon the then L.M. & S.R. system, using rigid-frame locomotives, had proved that a 1-in-200 cant gradient could be safely accepted at low speeds; Mr Turner would be interested to have the Authors' opinion on the application of such a cant gradient in the type of connexion to which he referred.

In trailing crossover roads between curved main lines, the cant gradient that could be "built-in" the connexion by means of two-level switches and thick-base chairs determined the relative levels of the running lines with the applied main-line cant. A cant gradient in the crossover of 1 in 200 was often necessary where the connexions were in close proximity to station platforms or tight overhead structures, and where a reduction in the main-line cant would involve a severe speed restriction on a high-speed curve.

Mr Maycock's principle of using the distance between bogie centres as a virtual transition had an interesting application when the versine on a chord equal to the bogie centres was measured at the toe of standard switches. It would be found that the versine at the toe of "D" switches, referred to as an example on p. 537, gave a resultant cant requirement of more than 3 inches at that point, the centre of the virtual transition, which compared unfavourably with the cant requirement of 2½ inches through the turnout radius of 21 chains, at the quoted speed of 29·5 miles per hour.

The substitution of "E" switches into the "D" heel chairs reduced the effective cant requirement at the toe of the switch to 2½ inches, thus enabling a constant cant deficiency to be attained throughout the turnout.

To carry the process a stage further, " E " switches fitted into " D " heel chairs with a 1-in-10 crossing gave a turnout lead closely approximating a transition, and by the use of a two-level switch and thick-base chairs in the turnout curve, a constant cant deficiency of 2½ inches could be attained throughout that transitional lead.

That procedure had been extended on the L.M.S.R. system to cover " long C " and " long E " switches, with a consequent speeding-up of traffic over main-line junctions and a saving in maintenance, by reducing the " throw " resulting from normal angles of side planing in standard turnouts.

Mr B. W. Wilson, of Johannesburg, South Africa, observed that the work described by the Authors, as conducted under the auspices of the Track Committee in Britain, was not dissimilar to certain studies with which he had been intimately connected in South Africa. Under South African conditions, with a railway gauge of 3 feet 6 inches and curvatures as sharp as 300 feet radius, the problems of curve design were possibly a little more intricate than in Britain. Certainly the safety aspect involving considerations of overturning of vehicles on curves was vitally important. The equivalent feature under British conditions no doubt justified the omission of any mention by the Authors of a criterion of safety against overturning, but it would be interesting to know, nevertheless, the value of the appropriate factor of safety. From data collected from various sources, but chiefly from the annals of the International Railway Congresses, Mr Wilson had found [1] that world practice seemed to adhere to a minimum factor of safety of about $q = 4{\cdot}0$, where q denoted the number of times the moment of resistance, provided by the weight of a vehicle, exceeded the moment of overturning, provided by the centrifugal forces. On the South African Railways, values of q as small as 3·0 (with flexible vehicles) were obtained on curves of certain radii on some classes of line,[2] but endeavours were being made in current revisions of maximum permissible speeds to bring that figure nearer 4·0.

A second factor, which seemed to have received rather scant consideration by the Authors, was that of economy, but there again the long mileages of single lines on the South African Railways, which had to cater for up and down traffic on a wide range of heavy gradients—traffic, moreover, with a preponderant proportion of freight services—provided conditions that were relatively more severe than in Britain and necessitated a degree of generalization in the selection of equilibrium cants which might not be desirable on the British Railways. Mr Wilson had succeeded in showing that, from the point of view of economy (having regard to the wear and tear on vehicle and track), the desirable value for the equilibrium speed V_e would be given by[1] :

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[1] " Superelevation and Maximum Speeds on Railway Curves." Thesis pub. by University of Illinois, 1940.

[2] E. M. Retief, " Applied Economics of Railway Deviations." Min. Proc. S.A. Instn Civ. Engrs, vol. 46 (1948), p. 35. *Discussion* by B. W. Wilson, p. 141.

$$V_e^2 = \frac{\Sigma pWV^2}{\Sigma pW} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where p denoted the frequency of any train of weight W and speed V. On the South African Railways, however, owing to the necessity for making speed concessions on curves in favour of passenger trains, equilibrium speeds were generally higher than was prescribed by that equation. Current practice was to determine the actual speeds of the various types of trains on representative curves of a given line, to convert those into equivalent cants, and thus derive cant-radius diagrams for trains of different categories; for example, up and down passenger trains and up and down goods on both steep and easy grades. A compromise was then necessary between the up and down trains for both passengers and goods, and finally a mean diagram for both steep and easy grades had to be determined by weighting conditions in favour of passenger trains.

The question of passenger comfort had received some attention in South Africa on rather similar lines to those described in Part I of the Paper.

A preparatory theoretical study had shown that on a flexible vehicle the horizontal centripetal acceleration acting on a passenger at maximum speed V_m on a curve of radius R could be expressed as :

$$\ddot{y} = \frac{V_m^2 - V_e^2}{R}\left[1 + \frac{\sigma}{1-\sigma}\cdot\frac{H-K}{H-K_o}\right] \quad . \quad . \quad . \quad . \quad (2)$$

in which σ denoted a factor defining the flexibility of the superstructure of the vehicle on its springs, and H, K, and K_o denoted dimensions relating respectively to the heights of centre of gravity of the superstructure and the undercarriage, and to the position of the centre of oscillation.[2] A typical value of σ for a passenger coach might be of the order 0·3, and the ratio $\frac{H-K}{H-K_o}$ tended to be greater than unity, making the value of the second term within the brackets of equation (2) of the order of 0·5. From that it would appear that the influence of the springing of a vehicle on the unbalanced radial accelerations could be important.

In 1940, Mr Wilson had initiated a series of tests to relate sensations of comfort or discomfort with values of acceleration $\ddot{y}$ and had selected for the purpose a Wickham trolley which to all intents and purposes could be regarded as a rigid vehicle. As such, flexibility could be discounted and accelerations (in feet per second per second) legitimately computed from:

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[1] See reference 1, p. 564.

[2] See reference 1, p. 564. For experimental background provided by tests of overturning vehicles on curves, conducted in 1937, see "Derailing Tests on South African Railways." *The Engineer*, vol. 166, Part 2, p. 581 (25 Nov. 1938).

$$\ddot{y} = \frac{V^2 - V_e^2}{R}$$

or

$$\ddot{y} = 2{\cdot}15\frac{V^2}{r} - 0{\cdot}767E \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where V denoted the speed in miles per hour, r the curve radius in feet, and E the cant in inches.

A code of sensations had been adopted, based on pure judgement, as followed:

TABLE 9.—CODE OF SENSATIONS

Sensation	Symbol
Very comfortable	*v.c.*
Comfortable	*c.*
Comfortable-uncomfortable . .	*c.–u.*
Uncomfortable	*u.*
Very uncomfortable	*v.u.*
Extremely uncomfortable . . .	*e.u.*

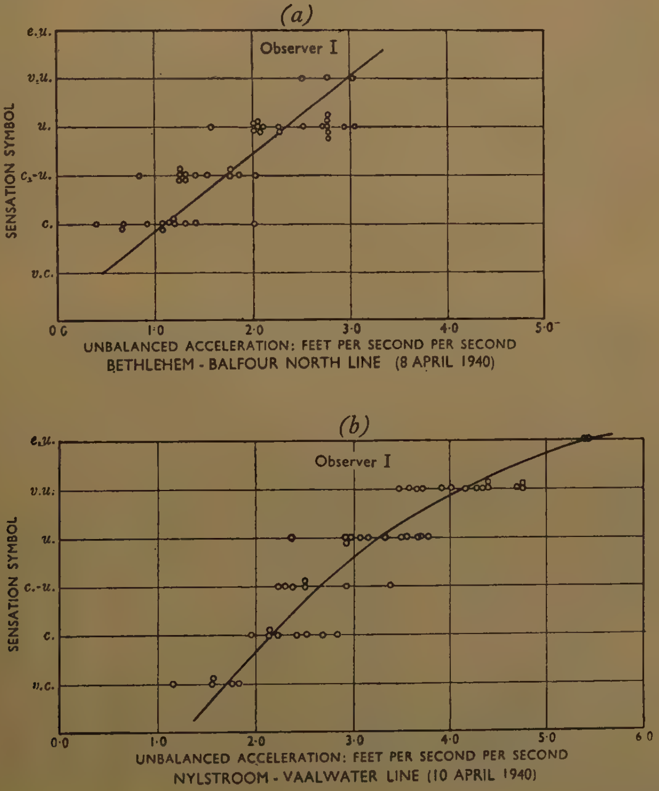
Two trips had been made at that time on the Bethlehem–Balfour North and Nylstroom–Vaalwater lines, the trolley being run at normal cruising speeds of about 35 to 40 miles per hour with occasional variations within a range of from 30 to 50 miles per hour. Curve radii varied from 8½ to 43 chains on the Balfour North line and from 7 to 40 chains on the Vaalwater line.

The practice had been simply to record the sensations felt in terms of the above symbols as the trolley negotiated the curves. No attempt had been made to discriminate between transitions and circular portions of curves, but care had been exercised to discount any sudden lurches arising from faulty track alignments. Speeds of the trolley had been read direct from the recording speedometer to give values of V, and cants E and radii r had been obtained later from track records.

The computed values of $\ddot{y}$ plotted against sensation symbols were shown in *Figs 18* (*a*) and *18* (*b*). The results, which reflected only Mr Wilson's observations in each case, showed considerable variance for the two lines. The disparities might have arisen from different personal perceptions of comfort on the two occasions, the standard on the Vaalwater line presumably having been conditioned by the sharp contrast between comfort and extreme discomfort, not experienced on the other line.

No further experiments had been conducted until 1947, when he and three other observers in a Wickham trolley had recorded their sensations on the curves of the Natal main line between Durban and Pietermaritzburg. The results were plotted in *Figs 19* and, despite considerable scatter, showed a consistent trend.

Figs 18

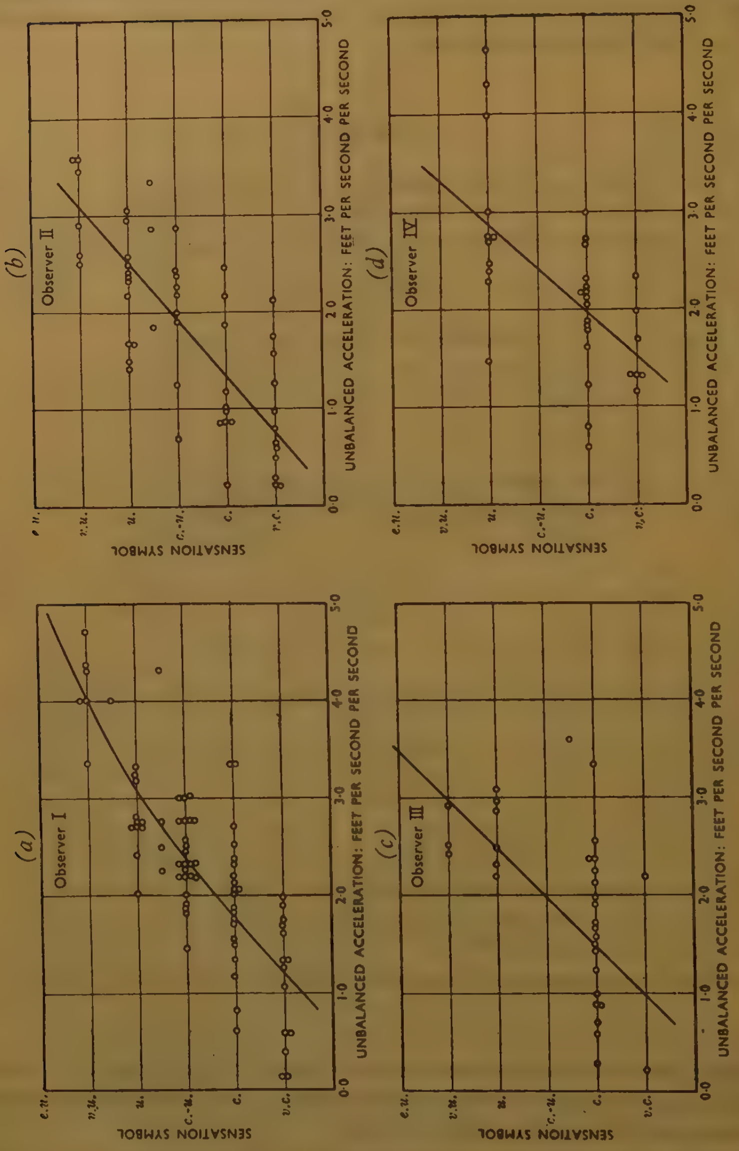


Radial Accelerations on Curves in Relation to Passenger Comfort

Taking the values of the acceleration $\ddot{y}$ where the mean lines of the diagrams intersected the borderline of discomfort (*c.–u.*), the following limits to the sensation of comfort were obtained.

The demarcating radial acceleration, taken as the weighted mean of the figures in Table 10, was 2·2 feet per second per second, and represented an unbalanced cant on 3-foot-6-inch-gauge track of 2·9 inches. The equivalent cant deficiency on standard-gauge track would be 3·9 inches. Allowing,

Figs 19



RADIAL ACCELERATIONS ON CURVES IN RELATION TO PASSENGER COMFORT. NATAL MAIN LINE

TABLE 10.—LIMITING RADIAL ACCELERATION FOR PASSENGER COMFORT

Line	Observer	Borderline value $\ddot{y}$: feet per second [2]	Number of observations
Vaalwater	I	2·66	48
Balfour North	I	1·70	45
Natal main	I	2·40	81
,, ,, ...	II	1·92	52
,, ,, ...	III	1·97	32
,, ,, ...	IV	2·42	36
Weighted Mean :		2·20	294

however, for the flexibility of coaching stock, equation (2) would modify (3) to

$$\begin{aligned}\ddot{y} &= 1{\cdot}5 \times 0{\cdot}767(\epsilon - E)\\ &= 1{\cdot}1D\end{aligned}$$

where ϵ denoted the cant in inches equivalent to the speed V, and D $(= \epsilon - E)$ denoted the cant deficiency in inches. Accordingly, for a limiting value of $\ddot{y} = 2{\cdot}2$ feet per second per second, the limiting cant deficiency would be about 2·6 inches for 3-foot-6-inch gauge and 3·5 inches for a standard gauge. The latter figure agreed very well with that adopted by the British Track Committee. In South Africa, making some allowance for imperfect alignment, it had been decided to adopt a maximum cant deficiency of 2 inches.[1] It might be mentioned that that figure and its equivalent for standard-gauge track were in good accord with the consensus of opinion expressed in engineering literature,[2] and particularly with American railway practice, which had adopted a maximum permissible cant deficiency of 3 inches.

The necessity for adopting an empirical formula for the reduction of cant deficiency with increase of cant or reduction of radius, mentioned in Part I of the Paper, diminished considerably if the factor of safety q was made a criterion as well as limiting cant deficiency D_m. That statement could perhaps best be explained by recourse to *Fig. 20*, in which curve AB represented the maximum speeds on curves of various radii such that the factor of safety q was never less than a minimum value, say 4·0. Curve FG was representative of overturning speeds (V_c), which bore an approximately constant ratio to the maximum speeds. Curve DE would represent the maximum speeds that could be allowed for comfort if a cant deficiency of D_m were not to be exceeded.

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[1] "Superelevations and Maximum Permissible Speeds on Curves." Research Circular No. 25.027, issued by the Office of the Chief Civil Engineer, South African Railways, 1948.

[2] See reference 1, p. 564.

The actual difference between V_m (safety) and V_c increased consider ably with radius, and travel on large-radius curves became ever safer wit increase of radius because of the larger marginal speed $(V_c - V_m)$. On flat curves, overturning was thus virtually impossible and it would be quit

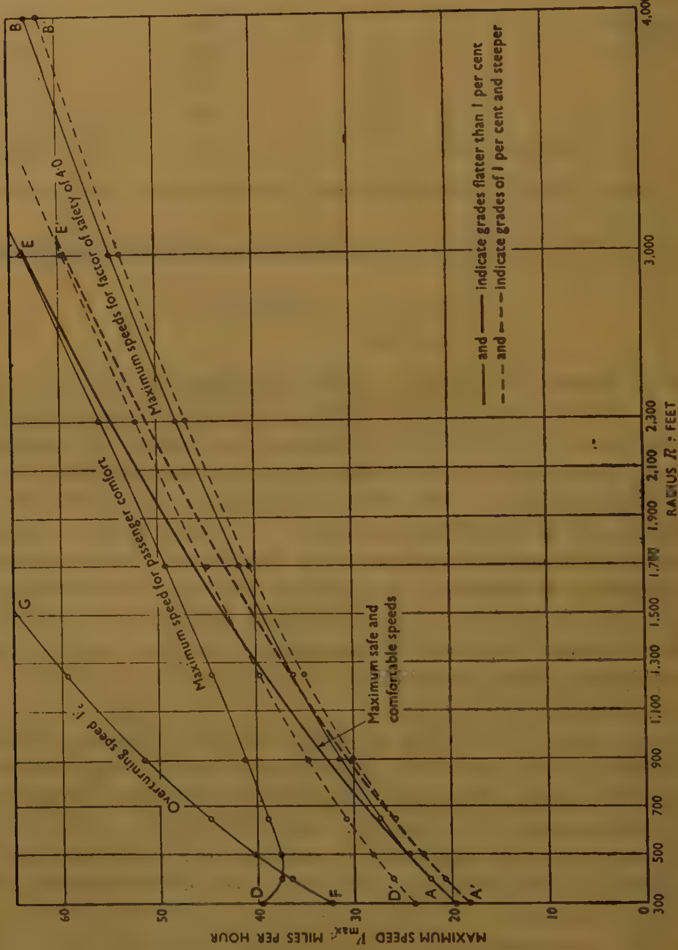

Fig. 20

MAXIMUM SPEEDS ON RAILWAY CURVES

safe to allow some reduction of the factor of safety q by adopting a relation ship for maximum speed V_m (safety and comfort) such as curve AE which was tangential to AB at A and asymptotic to DE at E. The relation ship AE gave the desired factor of safety on sharp curves where it was mos needed and the desired comfort on flat curves: in the intermediate rang the cant deficiency fell off from the maximum permissible to the exten that the adopted curve AE fell below DE in *Fig. 20*, but the gradation depended essentially upon the composite curve AE and the condition that it should merge with the confining curves AB and DE. That procedure, in essence, had also been adopted in South African Railway practice.[1]

Mr Loach, in reply, observed that, contrary to the opinions of most others, Mr Turner had suggested that the limiting permissible steepness of cant gradient might with advantage be fixed at 1 in 200 instead of 1 in 300, and in support of his statement he had mentioned tests with rigid-frame locomotives. Mr Loach stated that, unless the information to be gained from the tests involving locomotives on artificial cant gradients showed clearly that a gradient steeper than 1 in 300 could be tolerated, he would be reluctant to depart from the limit of 1 in 300 already agreed. It should be borne in mind that locomotives with 4–6–2 and 2–6–4 wheel arrangements might give worse conditions than a 0–8–0, for, though the fixed wheelbase was less, the redistribution of weights on the end carrying units could be appreciable because of the unequal loading imposed on their sides by the main frame. This became important on occasions when the centering forces in the unit were also appreciable.

Mr Loach thanked Mr Wilson for his very interesting contribution; it was pleasing to learn that the thoughts of the South African engineers had developed along the same lines as those described in the Paper. It seemed to be unfortunate, however, that Mr Wilson had confined his tests to runs on a Wickham trolley, because the rather rough ride, in contrast to that in a coach, would distract attention from or possibly mask somewhat the sensations to be considered; it is suggested that the scatter of the results may be attributed to it. Afterwards Mr Wilson had assumed factors to take account of the springing of the vehicles, factors which might be misleading if not handled carefully. Mr Loach considered it important for sensations to be recorded in the same surroundings and comfort as normally occur in travel and so he suggested that Mr Wilson's investigation would have been more complete if, finally, tests had been made with observers sitting in a coach and experiencing all the factors normally present in travel.

Mr Maycock agreed with Mr Turner that changes of curvature, cant, and deficiency of cant should be avoided so far as possible, but that it was sometimes necessary to make such changes on main lines through junctions and that the reduction of cant might, in some cases, necessitate speed restrictions.

One was reluctant to sacrifice speed or to vary curvature and cant on main lines on account of less important trailing connexions having negative cant, but in extreme cases it might be necessary to do so. A maximum allowable amount of negative cant had not been suggested in the Paper; all that had been said was that negative cant exceeding 3 inches at switches and common crossings and 2½ inches at obtuse crossings was undesirable. It was appreciated that in certain cases a somewhat higher negative cant might be justified, but Mr Maycock considered that the use of higher amounts of negative cant should not be generally encouraged.

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[1] See reference 1, p. 569.

Mr Maycock appreciated the advantages to be gained by the use of the "long C, D, and E" switches, referred to by Mr Turner, in reducing the momentary thrust at the point of divergence and in giving a transition effect through the switches. The treatment of the subject of speeds through turnouts in the Paper had been intended, however, to provide a simple general rule applicable to standard leads and that had been based on the radius, cant, and deficiency of cant of the turnout curve without taking account of possible variations in the design of switches. Where a suitable transition of sufficient length could be provided at the entry to a diverging line a somewhat higher maximum speed could be allowed. In such a case the diverging line could be treated as a main line for the purpose of assessing maximum speed.

MARITIME AND WATERWAYS ENGINEERING DIVISION MEETING

8 April, 1952

Lieutenant-Colonel R. H. Edwards, M.I.C.E., Chairman of the Division, in the Chair.

The following Paper was presented for discussion and, on the motion of the Chairman the thanks of the Division were accorded to the Author.

Maritime Paper No. 20

"Model-Tests on the Bellmouth-Spillway and Outfall of the Fassideri Project"

by

Edwin Samuel Crump, C.I.E.

SYNOPSIS

In the Fassideri project, works designed to protect the earthen dam from damage by floods comprised a bellmouth-spillway to discharge into an existing diversion-tunnel, and a steeply graded outfall channel linked to the tunnel exit through a chute and a stilling-basin fitted with Rehbock deflectors. As against an estimated high-flood discharge of 150 cumecs, the spillway was designed to pass a maximum discharge of 300 cumecs before water rising in the bellmouth would submerge the spill-weir and cause a dangerously rapid rise of reservoir-level.

With the object of obtaining a reliable forecast of prototype behaviour, various tests were carried out on a model structure, built to a scale of one inch to one metre. All parts of the model upstream of the tunnel exit were made up from transparent perspex sheets. The irregular supply-channel of the prototype was replaced in the model by a deep circular basin enabling tests to be made with the bed at, or below, designed level. To correct for scale-effect in the frictional losses, the tunnel of the model was given a steeper slope and shorter relative length than that of the prototype.

Tests were made with four arrangements: with a deep or shallow supply-channel, and, in each case, with and without a divide-wall inserted to kill the swirl in the supply channel. Without a divide-wall, the swirl caused the formation of a single vortex in the bellmouth: with a divide-wall, forcing the supply-stream to bifurcate, a pair of contending vortices was formed. Tests showed that with a divide-wall, a deep channel would safely pass 316 cumecs, but that without it, 241 cumecs would cause a very rapid and dangerous rise of water-level. With a shallow channel as designed and a divide-wall, a discharge of 300 cumecs would result in the designed reservoir-level being exceeded by the negligible amount of 0·02 metre; without the divide wall this would increase to only 0·26 metre.

With Rehbock deflectors in place, the standing wave was always contained within the stilling-basin, but excessive turbulence caused heavy spilling, which required increased freeboard. With the deflectors removed, a discharge of 272 cumecs swept the standing wave right out of the basin, suggesting the desirability of either raising the outfall channel or lowering the floor of the basin.

Piezometer measurements indicated that cavitation might occur in the throat of the bellmouth, and that low pressures in this region might be due to the existence of a *vena contracta* caused by imperfect shaping of the bellmouth.

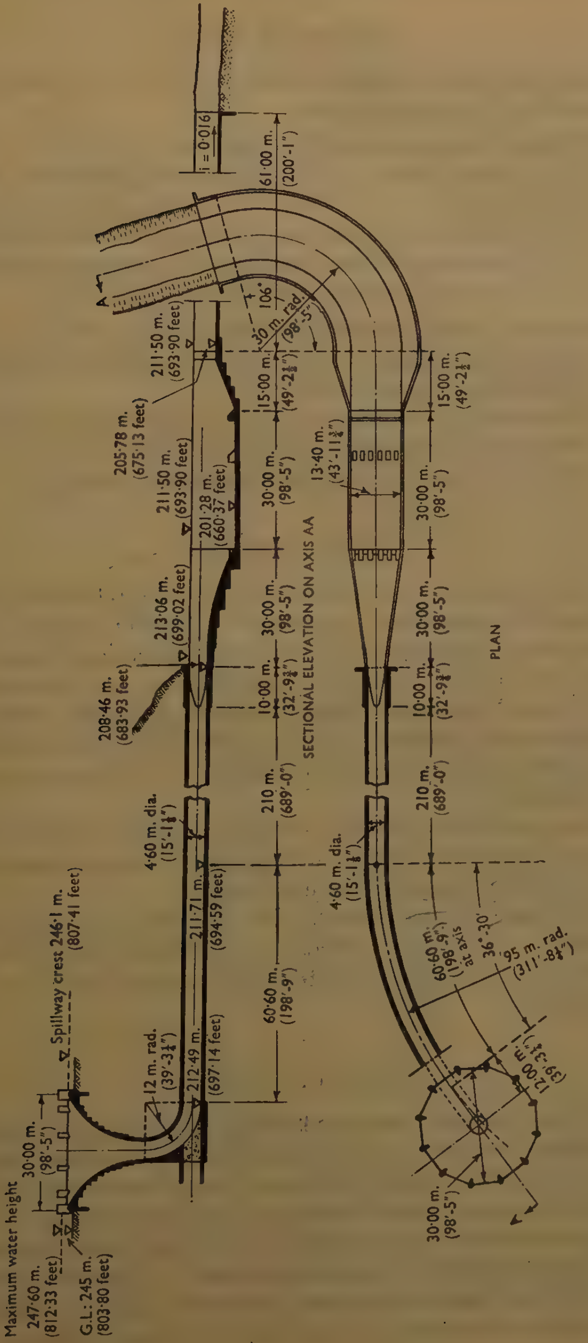
INTRODUCTION

THE model-tests described in the Paper were carried out by the Author under the direction of Sir Claude Inglis, Director of the Hydraulics Research Station of the Department of Scientific and Industrial Research, at the request of Sir William Halcrow & Partners, acting on behalf of the Greek Sanitation and Hydraulic Works Construction Company, whose Chief Designing Engineer, M. Th. Michalopoulos, was responsible for the design of new works in connexion with the Fassideri project on the river Kifisos near Athens. This scheme, now under construction, provides for the storage, for irrigation purposes, of 3,270,000 cubic metres of water by means of an earthen dam 33·5 metres high. Whilst the maximum river flow in normal years is estimated at 150 cumecs it was deemed advisable to design the spillway to have a maximum capacity of twice this discharge.

FEATURES OF PROTOTYPE DESIGN

The works, designed to deal with a flood of 300 cumecs are shown in *Figs 1.* The new bellmouth-spillway is located with its axis intersecting that of an existing diversion tunnel built before 1940. A short length of the tunnel is to be abandoned and plugged off, the remaining portion being embodied in the new design. The crest of the spillway is formed by twelve sub-weirs, each 7 metres wide between the parallel vertical sides of twelve pear-shaped guide-vanes. The crests of the weirs are cylindrical (radius = 0·894 metre) so that with an ideal radial approach, the flow over the weirs would be two-dimensional and not radially convergent. The bellmouth proper is meticulously designed as a paraboloid of revolution conforming to the shape that would be assumed with a discharge of 300 cumecs by a symmetrically convergent nappe of increasing thickness falling freely in air, from a conical lip of 10·80 metres radius inclined at 30 degrees to the horizontal. Neglecting frictional losses, the Chief Designer's calculations indicate that this ideal nappe would converge to form a solid jet of diameter somewhat less than that of the throat (and tunnel) at a point about 1·3 metre above the designed level of the throat. A short straight neck 1·22 metre in length connects the throat of the bellmouth to the upper end of the 12-metre-radius elbow. Downstream of the elbow, the tunnel has a uniform slope of 0·01436 and a total length of 280·6 metres, of which the first 60·6 metres is curved to a radius of 95 metres. To assist in the dissipation of residual energy, the stilling basin is provided with three rows of Rehbock deflectors, only the first two being dentated. The curved outfall-channel has a steep bed-slope of about 1 in 60, thereby ensuring hypercritical flow at all stages of discharge: the head of the channel therefore behaves as a free-fall weir, the modular action of which determines the level of water surface and energy-lines in the stilling-basin, and screens the basin from the effects of curvature in the outfall channel.

Figs 1



Original Design of Spillway, Tunnel, and Outfall

OBJECTS OF TESTS

The main points upon which it was hoped to arrive at a definite decision from the results of model-tests were :—

(1) Ability of the designed spillway to pass :
  (*a*) a normal flood of 150 cumecs with part-bore flow in the tunnel : and
  (*b*) a maximum discharge of 300 cumecs without endangering the safety of the earthen dam by serious encroachment upon the designed freeboard of 1·90 metres.

(2) Adequacy of the designed stilling-basin to fulfil its purpose of dissipating residual energy by the formation of a standing wave at the upstream end of the basin.

(3) The necessity, or otherwise, of providing ventilating ducts in the tunnel, either for relieving pressure in airpockets, or for admitting air to prevent or mitigate cavitation-effects.

SCALE OF MODEL

The scale suggested by the Chief Designer was 1/40, which required a model about 10 metres long. A scale of 1 : 39·37 or 1 inch : 1 metre was finally adopted. All prototype plans and calculations being in metric units, the convenience of this scale was obvious ; not only could dimensions for the model be taken directly from the prototype designs, but the measurement of all quantities of linear dimensions (that is to say, gauge and manometer readings) made in inches on the model represented metres on the prototype.

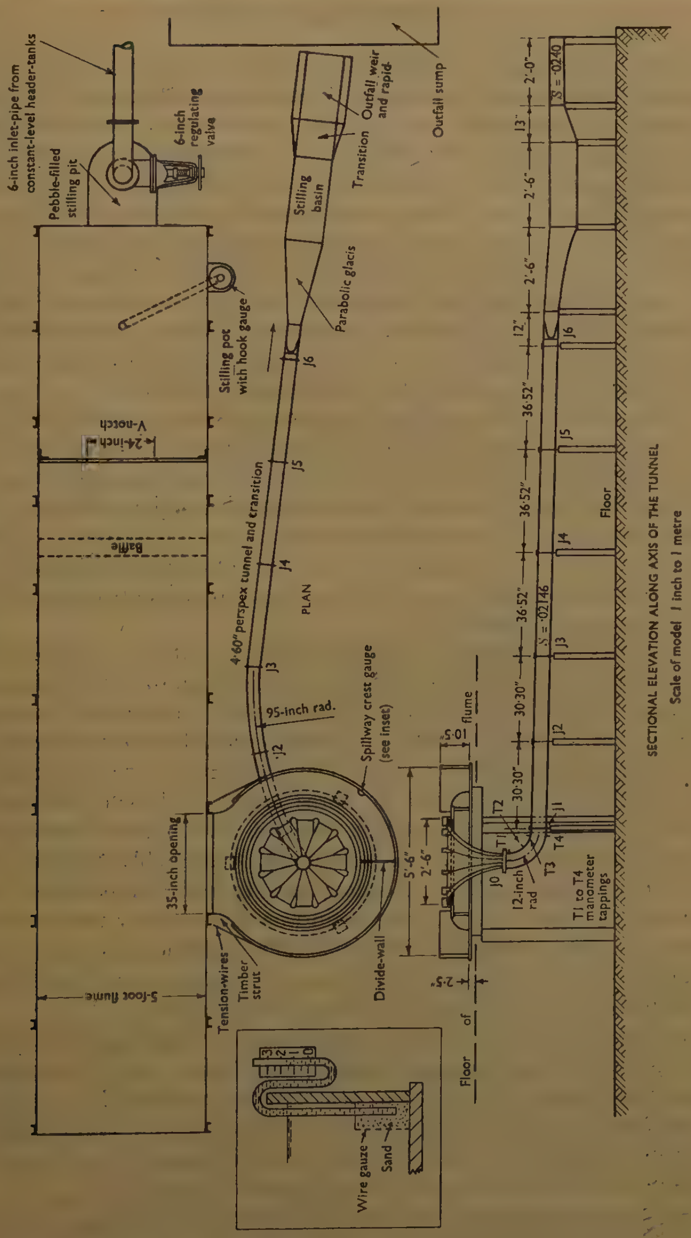
The main purpose of the investigation being to predict prototype performance, the results of the model-tests have been presented in terms of prototype behaviour expressed in metric units.

With the scale adopted, Froude's law requires the prototype discharge to be $(39{\cdot}37)^{5/2} = 9{,}726$ times that of the model, so that a discharge of 1 cusec in the model represents a prototype discharge of $(39{\cdot}37)^{5/2} \div (3{\cdot}281)^{3}$ $= 275{\cdot}4$ cumecs.

LAY-OUT OF MODEL

The lay-out of the model in relation to the relevant portions of the testing unit is shown in *Figs 2.* The unit comprises a circulating-pump o 1·5-cusec maximum capacity delivering into a capacious constant-leve header tank supplying, through a 6-inch pipe and regulating valve, a stead discharge to an elevated steel flume, 5 feet wide, which is fitted with number of plate-glass observation windows. A standard V-notch, 2 inches wide, cut from ¼-inch brass plate with $\frac{1}{16}$-inch square edge an $\frac{3}{16}$-inch chamfer, was fitted opposite the first of these windows.

Figs 2



SECTIONAL ELEVATION ALONG AXIS OF THE TUNNEL

Scale of model 1 inch to 1 metre

LAY-OUT OF MODEL

Discharges were computed from Barnes's empirical formula :

$$Q = 2{\cdot}48\, H^{2{\cdot}48} \text{ cusecs } (H \text{ in feet}).$$

The head H was measured by hook-gauge and vernier in a stilling-pot connected to the floor of the flume. Further down the flume, for feeding the model, another window was removed to provide an entrance 35 inches wide between two vertical stanchions. The perspex bellmouth, which was fitted with a ¼-inch perspex flange at designed floor-level (1·1 inch below weir-crests), was screwed down into a rebate let into the upper floor of a circular timber basin of 66 inches diameter and 10·5 inches depth. The main lower floor of the basin was 6·4 inches below the upper (designed) floor, so that under maximum-discharge conditions it gave a depth of flow of 9 inches. The model freeboard was thus 1·5 inch as compared with 1·9 metre for the prototype dam. The two floors of the basin were connected by a circular wax ring, rounded to a radius of 5 inches.

At the position shown in *Figs 2* a syphonic N-gauge made from ½-inch glass tubing was hooked over the wall of the basin, the meniscus of the outer U-limb being read from a scale adjusted to read zero with the water surface in the basin at average crest-level of the twelve sub-weirs. Precision in observing the gauge-reading G was greatly improved by burying the inner end of the N-gauge in a wire-gauze pot filled with sand, thereby damping oscillations that were otherwise found to vitiate readings taken on the higher ranges of discharge.

The substantial weight of the basin and its contents was transmitted through the radially jointed floor-boards of the basin to a stiff circular ring made from 4-inch-by-½-inch steel. With a view to minimizing strain and distortion, the radius of the circle of contact was adjusted to pass through the centre of gravity of sector-shaped elements with the basin fully loaded. The steel ring was supported by three 4-inch-by-4-inch wooden posts capped by adjustable hardwood fox-wedges. With this " three-point " support, precise levelling of the weir-crests was readily effected. As a further precaution against distortion, and to resist unbalanced water-pressures, the kerfed paling of the basin was embraced by two steel cables anchored to the entrance stanchions and tightened by turn-buckles. Tension in the cables was resisted by wooden struts screwed to the paling and shaped to fit snugly into the webs of the stanchions.

The perspex bends and tunnels were made up in sections and jointed by ¼-inch perspex flanges having twelve ¼-inch bolts. For convenience of reference the flanged joints may be considered as numbered from J0 to J6, the first being that at the upper end of the 12-inch elbow, and the last that at the upstream end of the exit-transition. Flanges J1 to J6 were supported on timber stools capped with adjustable fox-wedges.

The perspex bellmouth was not a " surface of revolution " as in the prototype, but was built up from twelve tapered panels cut to template and bent in a mould to the shape calculated by the Chief Designer. The cross-section of the throat 18·1 inches below crest-level was therefore dodecagonal. This section being 1·22 inch higher than the top of the elbow a flanged transition of this length, tapering from dodecagon to inscribed circle was inserted between bellmouth and elbow. Thus the splay at the corners of this transition was: $2{\cdot}3\,(\sec 15° - 1)/1{\cdot}22 = 0{\cdot}0665$, or 1 in 15 only.

With the object of obtaining Froude-law similitude of discharge with uniform part-bore flow in the tunnel, and again at the all-important stage of maximum capacity, the perspex tunnel of the model was given a steeper slope and shorter relative length than that of the prototype. These departures resulted in an invert-level at tunnel exits of 208·57 in the model and 208·46 in the prototype. With this small difference of 0·11 in levels, the model, from tunnel exits onwards, was an exact replica of the prototype except that, for reasons already stated, the outfall channel was made straight instead of curved, and—in order to simulate prototype depths of flow—was given a slope 50 per cent greater than the prototype.

The three Rehbock deflectors of the stilling-basin were made easily removable, and the test result analysed hereafter was made with all deflectors removed from the basin.

For observing pressures in the elbow J0–J1 four piezometric tappings were provided as follows:

T1 : on the concave spine, a little below J0.
T2 : „ „ „ „ midway between J0 and J1.
T3 : „ „ „ „ a little above J1.
T4 : „ „ convex belly, opposite T3.

These tappings could be connected either to a U-tube manometer for measuring vacuum pressures or pressures in air-pockets, or, alternatively, to one of two water-manometers placed side by side for measuring absolute and relative pressures at the opposed tappings T3 and T4. For the sake of clarity, readings of the former are indicated by the prefix *A* (Air) and of the latter by the prefix *W* (Water).

Supply Channel in Prototype and Model

In the prototype design, the spillway is located on gently sloping ground on the left flank of the dam. A supply channel giving access to the spillway will be formed by excavating a shallow basin of somewhat irregular shape with a level bed 1·10 metre below spill-weir crest-level. The centre of the spillway shaft will be about 22 metres from the line of intersection of the bed of the basin with the 1-in-2½ side-slope of the river-valley. On the landward side, the toe of a 3 : 1 side-cut will be nowhere less than 28 metres from the centre of the shaft.

In the model, by reason of the space and weight involved, it was considered impracticable to reproduce the topographical features of the prototype. To do so would have called for a second *ad hoc* model of much smaller size. With the model-arrangement already described, it was considered that a deep moat of low velocity would give conditions of flow approximating to the ideal of a purely radial approach devoid of initial swirl; whilst by filling the moat with fine gravel to the designed level of the upper floor and reproducing the valley side-slope of the prototype, it was considered that the behaviour of the model could be accepted as offering a safe prediction of that of the prototype as designed by the Chief Designer, since the wider entrance to the basin of the prototype would undoubtedly result in a weaker swirl in the basin than the relatively narrow entrance of the model.

Tests were made both with and without gravel filling.

BREAKDOWN FROM "FREE" TO "FULL-BORE" FLOW

In a tunnel of uniform section and slope, and of sufficient length for uniform flow to become established, the abrupt breakdown from part-bore flow with a "free" surface at atmospheric pressure, to "full-bore" flow with the tunnel under pressure, is a phenomenon well known to hydraulic engineers. It is due to the fact that, with an arched roof, as the depth of flow increases, the discharge attains the critical maximum value before the tunnel runs full. If the tunnel is called upon to pass a discharge very slightly greater than this critical value, it can only do so by running full-bore with a hydraulic-gradient steeper than the slope of the tunnel, so that the breakdown is attended by the expulsion of air from the tunnel, followed by afflux of the free surface at tunnel entrance.

In general, Manning's formula gives the discharge as:

$$Q = VA = \left[\frac{A^{\frac{5}{3}}}{P^{\frac{2}{3}}}\right] \times \left(1{\cdot}486\,\frac{S^{\frac{1}{2}}}{N}\right);$$

where A denotes sectional area of stream;
P ,, wetted perimeter;
S ,, hydraulic gradient;
and N is the coefficient of roughness.

In a tunnel flowing part-bore at uniform depth, the hydraulic gradient is identical with the constant tunnel-slope; so that breakdown occurs if and when the expression $\left[\frac{A^{\frac{5}{3}}}{P^{\frac{2}{3}}}\right]$ attains the maximum value. In the present case it can readily be shown that the expression has a maximum value of $2{\cdot}129\ r^{\frac{8}{3}}$ when the air-space subtends an angle of 58 degrees, where r denotes the radius of the circular section of the tunnel; whereas when the tunnel is full (that is to say when $A = \pi r^2$ and $P = 2\pi r$) its value is only $1{\cdot}979 r^{\frac{8}{3}}$.

Thus, the critical discharge Q_{max} at which breakdown occurs on a rising discharge is given by the equation:

$$Q_{max} = 2{\cdot}129 r^{\frac{8}{3}}(1{\cdot}486\ S^{\frac{1}{2}}/N)$$

where S denotes the tunnel slope; whereas, with full-bore flow, the discharge is given by the equation:

$$Q_{full} = 1{\cdot}979 r^{\frac{2}{3}}(1{\cdot}486\, f^{\frac{1}{2}}/N)$$

where f (steeper than s) denotes the hydraulic-gradient.

If the two discharges are the same, that is to say, if breakdown is attended by no rise in discharge, equating the two values of Q gives $2{\cdot}129S^{\frac{1}{2}} = 1{\cdot}979f^{\frac{1}{2}}$, that is to say, $f = 1{\cdot}158S$, showing that the afflux at tunnel-entrance will be $(0{\cdot}158S)$ times the length of the tunnel.

The Chief Designer, using a metric formula with which the Author was not familiar, computed the value of Q_{max} as 173·2 cumecs, that is to say, it exceeded the normal high flood of 150 cumecs. Comparing his result with the formula for Q_{max} given above, it was found that a value of $N = 0{\cdot}01358$ was required to give the same result, namely:

$$Q_{max} = 2{\cdot}129\ (7{\cdot}55)^{\frac{2}{3}}(1{\cdot}486(0{\cdot}01436)^{\frac{1}{2}}/0{\cdot}01358) = 6{,}100 \text{ cusecs}$$
$$= 173{\cdot}2 \text{ cumecs.}$$

The above value of N, considered reasonable for a well-moulded concrete surface, was used by the Author in all calculations relative to frictional losses in the prototype tunnel.

Frictional Losses in Prototype and Model

The application of Manning's formula:

$$V = \frac{1{\cdot}486}{N} \,.\, R^{\frac{2}{3}} \,.\, f^{\frac{1}{2}}$$

(where V denotes velocity in feet per second;
N is Kutter's N in British units;
R denotes the hydraulic radius in feet;
and f ,, the energy gradient due to frictional losses)

to a prototype m times larger than the model, leads to the relationship:

$$(V_P/V_M) = (N_M/N_P)\ (R_P/R_M)^{\frac{2}{3}}(f_P/f_M)^{\frac{1}{2}}$$

Froude similitude requires: $(V_P/V_M) = m^{\frac{1}{2}}$; and for similar cross-sections

$$(R_P/R_M) = m\text{; so that}$$
$$(f_M/f_P) = m^{\frac{1}{3}}\ (N_M/N_P)^2$$

Putting
$m = 39{\cdot}37$; N_M (for perspex) $= 0{\cdot}0090$ and N_P(for cement) $= 0{\cdot}01358$:

$$(f_M/f_P) = 1{\cdot}494$$

showing that in spite of its much greater smoothness, relative frictional losses in the tunnel of the model are nearly 50 per cent higher than those in the prototype. Thus, by giving the tunnel of the model a slope of 0·02146 (that is to say, 1·494 times steeper than that of the prototype), similar conditions of flow, throughout the whole range of part-bore flow, were ensured.

In the higher stages of discharge, with the tunnel running full-bore, in view of the limited freeboard provided in the prototype, dangerous conditions would set in as soon as the free-fall discharge of the spill weirs became affected by drowning. With cylindrical weirs, this drowning effect begins when the water-surface in the bellmouth rises to weir-crest level (246·10). At this stage of discharge, which may be designated the " maximum safe capacity " the Author's assumption—subsequently verified in model-tests—was that at tunnel exit the full-bore jet would acquire a free surface (at atmospheric pressure) precisely at the upstream end of the gently divergent transition. At this section, cradled by the transition and aerated only at its crown, it retains the characteristic features (such as the distribution of pressure and velocity) of the prismatic flow of the tunnel. Its residual kinetic energy is that of the tunnel flow, and the working head is correctly measured, not from the centre of the orifice (as in the case of a jet leaping into free air and aerated all over its perimeter) but from the soffit of the tunnel. The matter is important for the reason that the kinetic energy of the issuing jet accounts for nearly 50 per cent of the working head.

In order to ensure Froude similarity at " maximum safe capacity " in prototype and model, the tunnel of the latter was shortened by 98·50 inches, thereby reducing the frictional losses in the ratio of 190·95 : 289·45 (tunnel length in the prototype, measured from the upstream end of the elbow to the upstream end of the exit-transition). With the frictional gradient of the model 1·494 times that of the prototype, the combined effect of size and length on total frictional losses was, therefore, to reduce the relative losses in the ratio of 1·494 × 190·95/289·45 = 0·9858 : 1. On the assumptions indicated, the relative performance of prototype and model, both expressed in prototype language, is shown in Table 1.

TABLE 1

	Prototype	Model
Weir-crest level . . .	246·100	246·100
Soffit-level at exit . .	213·204	213·385
Working head	32·896	32·715
Losses of head expressed as multiples of $V^2/2g$		
Losses in bellmouth . . .	0·1000	0·1000
Additional losses in bends	0·2500	0·2500
Frictional losses . . .	0·8694	0·8571
Kinetic energy at exit .	1·0000	1·0000
Total losses	$2{\cdot}2194 \times \frac{V^2}{2g}$	$2{\cdot}2071 \times \frac{V^2}{2g}$

Equating total losses to working heads, the value of $V^2/2g$ in the two cases is identical, namely 14·822 metres. This gives $V = (2g \,.\, 14{\cdot}822)^{\frac{1}{2}} = 17{\cdot}053$ metres per second, and $Q = AV = 16{\cdot}619 \times 17{\cdot}053 = 283{\cdot}4$ cumecs compared with the assumed maximum value of 300 cumecs.

EFFECT OF UNDERESTIMATION OF KUTTER'S N

In considering the value of $N = 0{\cdot}01358$ used in assessing prototype performance and designing the model, the consulting engineers expressed their opinion as practising engineers that so low a value would only be attained by costly special methods of construction. They had frequently found by subsequent measurement on actual works that values less favourable even than 0·014 might be obtained.

Confronted with so authoritative an opinion, it was felt necessary to examine the effect on the estimated "maximum safe capacity" of increasing the value of N by 3·07 per cent, that is to say, to the value of 0·014. On the Author's assumptions, the working head is given by:

$$H = (1{\cdot}350 + kN^2)\, V^2/2g$$

where H denotes the working head and k is a constant required to give the friction term (kN^2) the value of 0·869.

For the given value of H(32·896 metres) the value of the discharge is:

$$Q = K(1{\cdot}350 + kN^2)^{-\frac{1}{2}}$$

where K is a second constant.

Logarithmic differentiation then gives:

$$\frac{dQ}{Q} = -\frac{kN^2}{1{\cdot}350 + kN^2} \cdot \frac{dN}{N}$$

Putting $kN^2 = 0{\cdot}869$ gives:

$$\frac{dQ}{Q} = -\,0{\cdot}392\,\frac{dN}{N}$$

so that an increase of 3·07 per cent in the value of N would decrease the calculated value of Q by only 1·20 per cent.

It may, therefore be inferred that small errors made in assessing the values of N for prototype and model would not seriously vitiate a prediction based on the results of model-tests.

DANGER OF WORKING WITH SPILL-WEIRS SUBMERGED

In considering the "maximum safe capacity" of the spillway, it is at once obvious that so long as the spill-weirs remain undrowned and discharge freely into the bellmouth, surface-level in the reservoir will be quite unaffected by anything that is happening in the bellmouth and tunnel. In these circumstances the crest gauge reading will be proportional to $Q^{\frac{2}{3}}$, so that for a small increment δQ in the discharge, the increment δG in gauge-reading is given by the equation:

$$\delta G = \tfrac{2}{3} \,.\, G \,.\, \frac{\delta Q}{Q}$$

For example, if $G = 1{\cdot}50$ when $Q = 300$ cumecs, then for a 1 per cent increase in discharge the rise would be only 0·010 metre.

On the other hand, when the weirs are completely submerged, it is the relationship between the discharge Q and the working-head $H = (G + 32{\cdot}9)$ that determines the value of G. Since Q is now proportional to $(G + 32{\cdot}9)$ the differential relationship becomes:

$$\delta G = 2(G + 32{\cdot}9) \,.\, \delta Q/Q$$

If, with the weirs submerged, the gauge-reading stood at $G = 1{\cdot}6$ (0·1 metre in excess of design) a further increase of 1 per cent in the discharge would result in a very dangerous rise of $\delta G = 0{\cdot}690$ metre—69 times greater than for the previous case.

The Fassideri Spillway has been designed on the assumption that a maximum discharge of 300 cumecs would be passed with the spill-weirs still working free-fall, with a metric weir-coefficient of 2·0 (corresponding to 3·622 in British units) and a gauge-reading G of 1·5 metre. With a free-board of only 1·9 metre, the above analysis serves to emphasize the danger involved in the spillway operating with the spill-weirs submerged.

DISCHARGE-TABLE FOR OUTFALL WEIR

As previously stated, the conditions of flow downstream of the standing wave in the stilling-basin are determined by "modular" action of the weir formed by the head of the outfall channel. Regarding this as a long-throated weir, the discharge Q for various values of the depth of flow d is found by applying the principle of "minimum energy", the general criterion for which is:

$$h = V^2/2g = A/2w\,;$$

where $h = V^2/2g$ = velocity head;
A denotes sectional area of flow; and
w ,, surface-width;

all taken at the "critical" section of the weir.

In the present case, working in metric units, this section has a bed-width of 13·4 metres with ½ : 1 side-slopes; so that:

$$w = 13{\cdot}4 + d$$

$$A = d(13{\cdot}4 + \tfrac{1}{2}d)$$

$$h = A/2w = \tfrac{1}{2}d(13{\cdot}4 + \tfrac{1}{2}d)/(13{\cdot}4 + d)$$

$$V = (2gh)^{\frac{1}{2}} = (gd)^{\frac{1}{2}}\,(13{\cdot}4 + \tfrac{1}{2}d)^{\frac{1}{2}}/(13{\cdot}4 + d)^{\frac{1}{2}}$$

and

$$Q = AV = g^{\frac{1}{2}} \,.\, d^{\frac{3}{2}} \,.\, (13{\cdot}4 + \tfrac{1}{2}d)^{\frac{3}{2}}\,(13{\cdot}4 + d)^{-\frac{1}{2}}$$

The total energy measured from the weir crest is $T = (d + h)$; and, measured from the floor of the stilling basin, is $T_y = (T + 4{\cdot}51)$.

Corresponding values of d, Q, T and T_y calculated from these expressions are shown in Table 2.

TABLE 2

d	Q	T	T_y
3·7	321·02	5·3498	9·8598
3·6	307·48	5·2094	9·7194
3·5	294·18	5·0688	9·5788
3·4	281·11	4·9280	9·4380
3·3	268·25	4·7870	9·2970
3·2	255·65	4·6458	9·1558
3·1	243·27	4·5044	9·0144
3·0	231·14	4·3628	8·8728

The Table was required for analysing standing-wave conditions in the stilling-basin.

EFFECT OF GUIDE-PIERS

The effect of guide-piers forming parallel side-walls for the cylindrical weirs was to vitiate the ideal conditions of radial flow assumed in designing the shape of the bellmouth. Flow over the weirs, when working free-fall, was not radial, but two-dimensional with flow lines parallel to the walls of the piers. The coalescence (at an angle of 30 degrees) of adjacent jets coming from the two sides of a pier, produced a marked concentration of flow where they met to form a smooth non-turbulent ridge of much greater depth than that of the main jets. The formation of twelve ridges in the wake of the piers was a conspicuous feature of model-behaviour. The deflexion of these ridges from radial planes (*Fig. 3*, facing p. 590) gave a clear indication of the strength of the swirl in the supply channel, and of the extent to which the swirl was subdued by inserting a thin metal divide-wall across the channel (*Fig. 4*, facing p. 530).

At stages of discharge attended by the formation in the bellmouth of a core of idling water, it seems reasonable to suppose that the amount of air entrained along the edge of the core would be substantially increased by the presence of corrugations in the surface of the descending nappe.

Obliquity of flow, in the stream approaching the weirs, sets up a strong action round the pier-noses and there is a marked end-contraction effect on the lee side of the pier; the effective length of the weir is reduced, and its discharge coefficient is smaller than it would be in the absence of guide-piers.

Conversely, action round the pier-noses results in unequal pressures on the two sides of the pier, so that there is a resultant force tending to overturn the pier. In resisting this force, the piers exert a turning moment (on the body of water) opposing the swirl in the supply channel. The strength of the vortex formed in the bellmouth is thereby reduced, with the beneficial result that the weirs become drowned at a higher discharge.

The effect of guide-piers, therefore, is to increase the "maximum safe capacity" at the cost of some reduction in the discharge-coefficient of the spill-weirs.

AIR ENTRAINMENT

Observation of the model showed the following sources of entrained air :

(1) At low discharges, before a core had formed in the bellmouth, the corrugated annular jet descending the sides of the bellmouth converged and united to form a single aerated stream flowing along the curved belly of the elbow. One half of the annular jet hugged the bend, whilst the other half, breaking away from the roof of the elbow, fell in a vertical curtain to join its mate. The falling curtain thus formed a partition completely separating the stream of air that flowed down the bellmouth and was carried, emulsified, along the tunnel, from the air-space overlying the water-stream of the tunnel. The entrained air was not retained, but escaped to the surface of the stream well before reaching the tunnel exit.

(2) At intermediate discharges, when a core of water stood in the bellmouth, air entrainment took place where the annular jet descending the sides of the bellmouth impinged upon core-water. The smaller the volume and height of the core, the more vigorous was the air entrainment, and, in the earlier stages of this phase, all entrained air was swept down into the tunnel. As the core rose and became larger, not only was less air entrained (the velocity of the jet being smaller) but, with a lazier core, an increasing fraction of the air entrained rose through the core and escaped from its surface. At the higher stages of discharge, before the core rose to weir-crest level, all air entrained escaped in this manner, none of it being carried into the tunnel.

(3) At high discharges, when the core nearly filled the bellmouth, with no divide-wall inserted to kill the swirl in the supply channel, this swir induced the formation of a powerful counter-clockwise vortex at the centr of the core (*Figs 5* and *6*). Depending upon the angular momentum of th swirl in the supply channel, the single vortex became so strong and pro duced so great an indraft of air, that, as will be seen later, the maximum safe capacity of the spillway was reduced to about 75 per cent of the valu it attained with a well-placed divide-wall in position. With a divide-wal inserted to kill the swirl, the tangential components of flow in the two halve of the supply channel caused the formation of two separate vortices o opposite sense in the core-water of the bellmouth. Air indraft with twi vortices was very much less than with a single vortex, and, with the inte play of the twins, the indraft of air was intermittent, resulting in a leisurel rhythmic rise and fall of the surface of the core (*Figs 7* and *8*). During th

lulls in the interplay there was no indraft of air whatever, and the whole tunnel was seen to be running full-bore with " black " water throughout its length. With one or other of the twins in the ascendant, the indrawn air was to be seen as a separate stream flowing along the roof of the tunnel.

ABSENCE OF ABRUPT AFFLUX

In their independent calculations, both the Chief Designer and the Author anticipated the occurrence of an abrupt change from " free " to " full-bore " flow, starting at the tunnel exit and running back to involve the whole of the tunnel and part of the elbow. It is of interest to record the fact that the model failed to exhibit any sudden afflux, either at their calculated value of 173·2 cumecs or at any other discharge. The rise up the bellmouth of a core of idling water actually took place gradually, with no abrupt changes, as the discharge was progressively increased ; and, on a failing discharge, retreated without the usual " hysteresis " effect. The reason suggested for this apparently abnormal behaviour is that the tunnel was too short for the uniformity of flow, tacitly assumed in the calculations, to become established before reaching the tunnel exit. With part-bore flow in excess of half-bore, the flow throughout the tunnel appeared to be hypercritical, the sectional area of the stream increasing, and its velocity decreasing, in the direction of flow. In these circumstances, if full-bore flow set in at the exit, the full-bore plug would not march upstream, as was supposed, but would extend only a short distance upstream to a point where it would be held stationary and under pressure by the greater momentum of the hypercritical jet impinging upon it. In fact, no such plug was formed, possibly because there was always a flow of liberated air in the roof of the tunnel.

ACTION IN STILLING-BASIN

In investigating the performance of the stilling-basin, tests were made both with, and without, the Rehbock deflectors of the prototype design. With the deflectors in, the standing wave was contained within the basin at all discharges. With high discharges, however, the turmoil in the basin was very severe : the first line of dentated deflectors was intermittently exposed, and the surface-divide marking the end of the surface roller was very near to the downstream end of the basin. Violent action in the basin was attended by massive upheavals resulting in heavy spilling over the side-walls. It was clearly demonstrated that to prevent spilling in the prototype it would be necessary to raise the side-walls by at least 1 metre. The observed action strongly suggested the advisability of increasing the capacity of the basin, either by lowering its floor, or by raising the level of the outfall weir. The effect of these two alternatives is examined below.

In a test run made with the supply channel clear of gravel and a divide-wall inserted, and with all deflectors removed from the stilling-basin, it was found that when the discharge attained a value of 272 cumecs, the hyper-critical jet descending the parabolic chute began to intrude into the stilling-basin. With the discharge held steady, the buffer of the standing wave was driven, at first quite slowly and later more rapidly, along the floor of the basin and up the reverse slope to the weir until it completely disappeared. It was found impossible to keep the wave in a stationary position anywhere in the basin by adjusting the discharge. As soon as the jet was strong enough to force an entry into the basin it was able to sweep the wave over the weir, thereby completely stultifying the purpose of the stilling-basin. *Figs 9* and *10*, facing p. 591, show the beginning and end of this action.

With a steady discharge of 272 cumecs passing over the spill-weirs, it was noticed that the surface of the core-water in the bellmouth rose and fell by about ± 1 metre from a mean position about 6 metres below spill-crest level, so that with a varying working head on the tunnel system the discharge entering the stilling-basin must have had a small slow pulse, although no such effect was revealed by behaviour in the stilling-basin.

In designing stilling-basins the Author has always considered it advisable, as a safety precaution, to neglect frictional losses on the chute. In this connexion it may be of interest to examine the observed behaviour

TABLE 3

	Model	Prototype	
		Case I	Case II
Q denotes discharge	272	300	300
V „ velocity at tunnel exit	16·367	18·051	18·051
$V^2/2g$ = kinetic head at tunnel exit	13·653	16·608	16·608
$q = Q/13{\cdot}40$ = discharge intensity	20·299	22·388	22·388
$c = (q^2/g)^{\frac{1}{3}}$ = critical depth of flow	3·476	3·711	3·711
T denotes total energy on weir-crest (see Table 2)	4·828	5·130	5·130
Add			
P denotes height of weir above floor of basin	4·510	*4·966	†5·017
$T_y = T + P$ = total energy downstream of wave	9·338	10·096	10·147
(T_y/c)	2·686	2·721	2·734
(T_x/c) taken from Table compiled by Author	7·334	7·689	7·826
$T_x = (T_x/c) \times c$ = total energy upstream of wave	25·490	28·532	29·039
F denotes fall from tunnel soffit to floor of basin	11·995	11·924	†12·431
Add			
$V^2/2g$ as above	13·653	16·608	16·608
Total energy at tunnel exit measured from floor of basin	25·648	28·532	29·039
Compare T_x as above	25·490	28·532	29·039
Frictional losses in chute	0·158	neglected	

(*) Crest of weir raised by 0·456 metre.
(†) Floor of basin lowered by 0·507 metre.

of the model and the two alternatives suggested in re-designing the prototype for a discharge of 300 cumecs. The alternatives suggested and examined below were:

(1) Raising the outfall weir by 0·456 metre.
(2) Lowering the floor of the basin by 0·507 metre.

For the simple case of a basin of rectangular cross-section having a level floor, the unique relationship that exists between the dimensionless ratios (T_x/c) and (T_y/c) is readily computed from first principles. Corresponding values shown in Table 3 have been taken from a Table compiled by the Author.

The above analysis of actual model-behaviour leaves a margin of only 0·158 metre to account for frictional losses in the chute. As against this small apparent loss, the Chief Designer's computed losses were 2·009 metres for a discharge of 300 cumecs, whilst an independent calculation by the Author, using Manning's formula with $N = 0{\cdot}01358$, gave a loss of 2·210 metres for a smaller discharge of 283 cumecs.

Although the test result supports the recommendation to neglect frictional losses in the chute, it is felt that an explanation is required to account for the large discrepancy between apparent and calculated losses. In spite of its high velocity, the smooth appearance of the jet traversing the chute suggests that actual losses may well be much smaller than those calculated from accepted formulae. A more plausible explanation is that the conventional method of evaluating the kinetic energy is not strictly correct; a more rigorous method would average the kinetic energy of elementary filaments, and, as is well known, if the velocity varies over the cross-section, the latter average may be substantially greater than the kinetic energy of the mean velocity.

RESULTS OF MAIN TESTS

The main object of model-tests was to predict what water level in the reservoir would be required to pass various discharges through the spillway, and, in particular, to ascertain the prospect of its passing 300 cumecs without endangering the safety of the earthen dam. Tests on the model, made with this object in view, were therefore of a very simple character, requiring only the simultaneous readings G of the spill-weir gauge, and of the V-notch gauge for computing the discharge Q. For this purpose, test runs were made with four separate arrangements of the model. These were:

Arrangement (*a*)—supply channel devoid of gravel filling and no divide-wall inserted to kill the swirl;

Arrangement (*b*)—as above with divide-wall inserted;

Arrangement (*c*)—supply channel filled with gravel to designed bed-level (1·1 metre below spill weir crest) and no divide-wall inserted; and

Arrangement (*d*)—as above with divide-wall inserted.

The standard position of the divide-wall was on the landward side of the spillway, directly opposite the centre of the 35-inch entrance, thereby dividing the circular basin of the model into two symmetrical halves. A preliminary trial having shown that the model behaviour was immune from any "hysteresis" effect, all tests were made with progressively increasing discharges.

With 1 cusec in the model representing 275·4 cumecs in the prototype, all results were expressed directly in terms of prototype performance in metric units. The results of five separate runs—those for (*d*) being repeated—are presented in Tables 4 to 8. Curves drawn through observed

TABLE 4

Arrangement (*a*)

Q	G	C	C'	K	Remarks
107·2	0·814	1·716	3·107	—	Weirs undrowned
144·1	0·964	1·790	3·241	—	,, ,,
194·3	1·179	1·785	3·232	—	,, ,,
211·2	1·324	1·630	2·952	—	,, ,,
231·6	1·494	1·491	2·700	—	,, ,,
241·0	2·794	0·606	1·098	40·47	Weirs drowned : single vortex in core

The decline in values of C is due to increasing swirl, and consequent starvation of the weirs by strong action at the pier-noses.

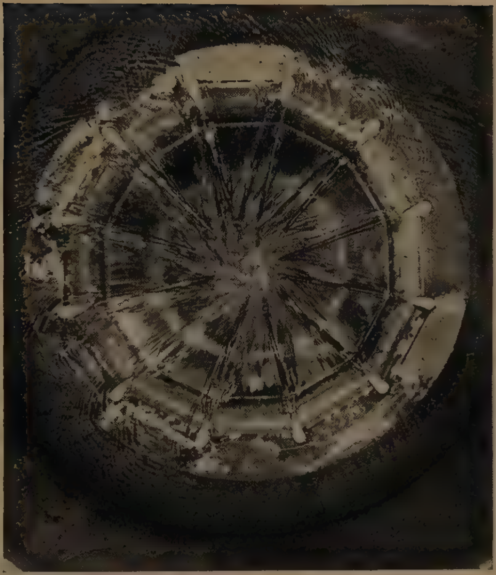
TABLE 5

Arrangement (*b*)

Q	G	C	C'	K	Remarks
109·0	0·794	1·811	3·280	—	Weir undrowned
144·9	0·924	1·917	3·471	—	,, ,,
187·6	1·064	2·008	3·638	—	,, ,,
212·8	1·144	2·044	3·703	—	,, ,,
233·0	1·194	2·100	3·800	—	,, ,,
252·5	1·264	2·087	3·782	—	,, ,,
272·0	1·334	2·074	3·757	—	,, ,,
288·0	1·384	2·078	3·765	—	,, ,,
303·3	1·429	2·087	3·780	—	,, ,,
311·0	1·474	2·041	3·700	—	,, ,,
316·4	1·494	2·035	3·688	—	,, ,,
316·4	1·894	1·426	2·583	53·80*	Weirs drowned : twin vortices in core.

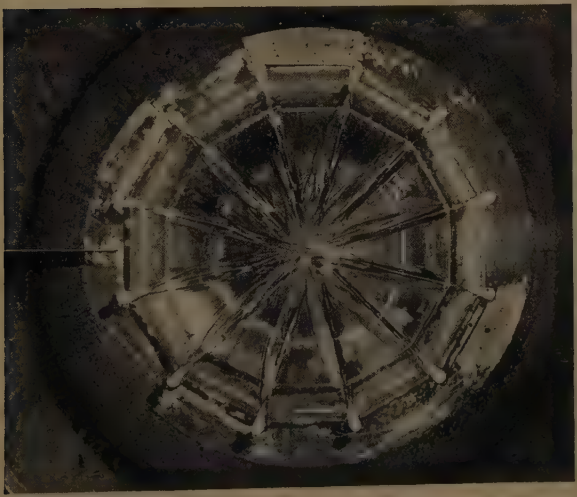
* This higher value of K compared with that of Table 4 is a measure of improve efficiency owing to suppression of the single vortex by the divide-wall.

Fig. 3



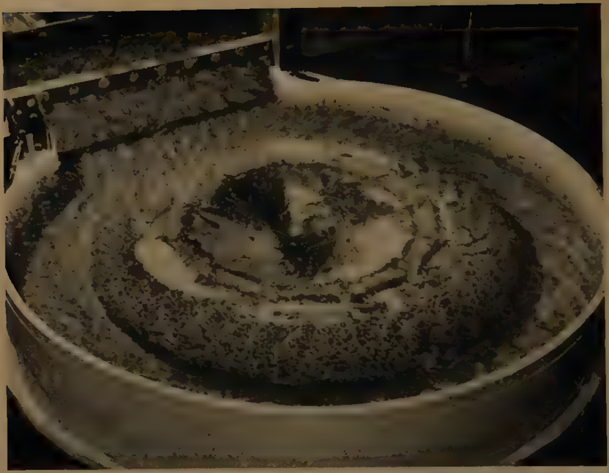
Discharge, 150 Cumecs; no Divide-Wall. Wake Ridges deflected by Swirl in Supply Channel

Fig. 4



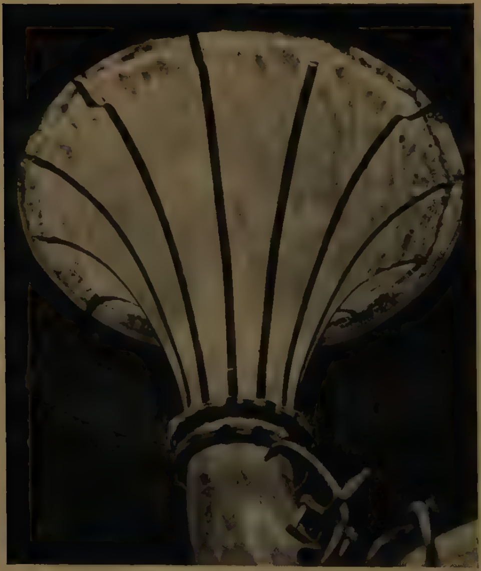
Discharge, 150 Cumecs; Divide-Wall in Place. With Swirl suppressed, Wake Ridges converge radially

Fig. 5



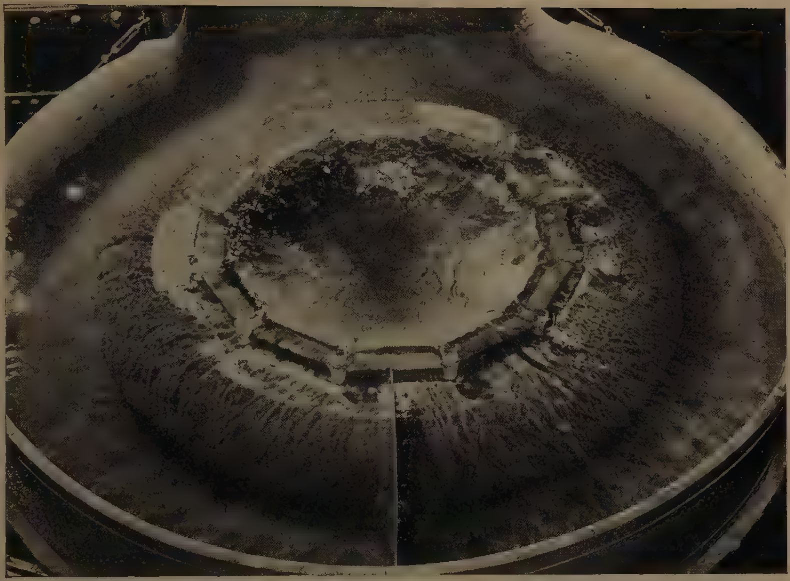
Discharge, 232 Cumecs ; no Divide-Wall. Single Vortex viewed from above

Fig. 6



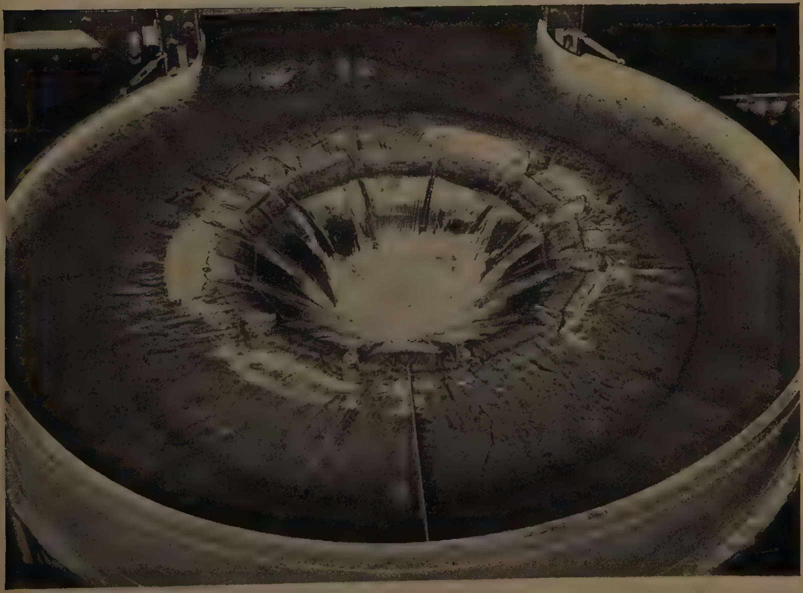
Discharge, 232 Cumecs ; no Divide-Wall (as in *Fig. 5*). Single Vortex viewe
from the Side

Fig. 7



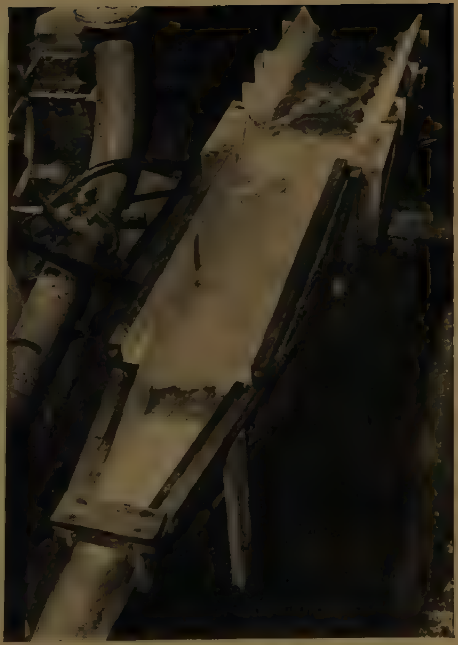
DISCHARGE, 316 CUMECS; DIVIDE-WALL IN PLACE. CORE-WATER IN BELLMOUTH AT TOP OF SWING

Fig. 8



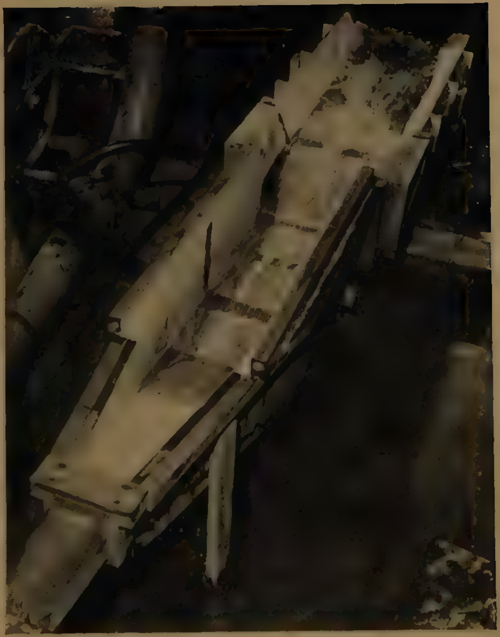
DISCHARGE, 316 CUMECS; DIVIDE-WALL IN PLACE (AS IN *Fig. 7*). CORE-WATER IN BELLMOUTH AT BOTTOM OF SWING

Fig. 9



Standing Wave beginning to Move Down the Stilling-Basin

Fig. 10



Stilling-Basin after the Standing Wave has been Swept Over the Outfall Weir

Discharge: 272 Cumecs

TABLE 6

Arrangement (*c*)

Q	G	C	C'	K	Remarks
274·0	1·609	1·578	2·858	—	Weirs undrowned
290·1	1·684	1·560	2·825	—	„ „
296·4	1·734	1·525	2·762	—	„ „
305·2	1·804	1·480	2·680	51·95*	Intermittent drowning: Single vortex in core.

* This higher value of K compared with that of Table 4 is due to a weaker whirl in the supply channel; a weaker single vortex; and less air indraft.

TABLE 7

Arrangement (*d*): first run

Q	G	C	C'	K	Remarks
107·1	0·774	1·849	3·347	—	Weir undrowned
126·0	0·864	1·842	3·335	—	„ „
144·3	0·949	1·834	3·322	—	„ „
163·3	1·014	1·877	3·400	—	„ „
190·7	1·124	1·880	3·404	—	„ „
211·5	1·204	1·880	3·404	—	„ „
231·8	1·284	1·871	3·390	—	„ „
253·4	1·349	1·900	3·443	—	„ „
271·3	1·409	1·907	3·453	—	„ „
288·5	1·464	1·914	3·466	—	„ „
301·5	1·534	1·852	3·355	—	„ „
315·0	2·094	1·221	2·210	53·4	Weirs drowned: twin vortices in core

TABLE 8

Arrangement (*d*): second run

Q	G	C	C'	K	Remarks
108·3	0·794	1·798	3·256	—	Weirs undrowned
142·0	0·934	1·848	3·347	—	„ „
189·8	1·124	1·870	3·390	—	„ „
232·9	1·284	1·880	3·406	—	„ „
253·7	1·344	1·912	3·464	—	„ „
272·7	1·404	1·925	3·487	—	„ „
288·1	1·464	1·910	3·460	—	„ „
303·1	1·529	1·884	3·412	51·80*	Weirs not fully drowned: twin vortices in core

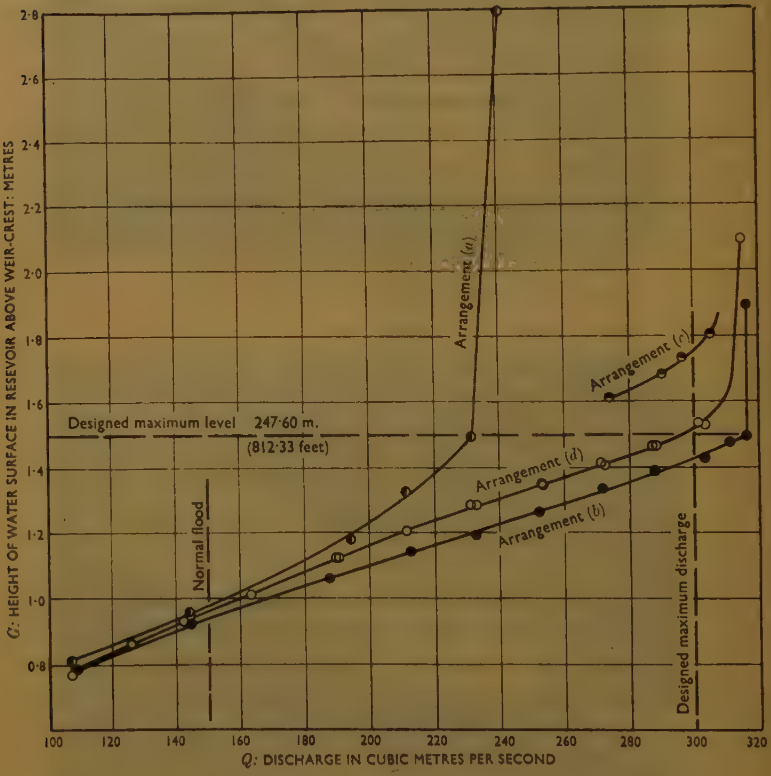
* This lower value compared with that of Table 7 is due to the fact that drowned conditions were not fully established.

results are shown in *Fig. 11.* The two runs made with (*d*) gave results so consistent that no distinction was made between them in plotting.

Headings in the five Tables have the following significance:

Q denotes observed discharge converted to prototype, in cumecs.
G denotes reading of N-gauge, in metres.
$C \;= Q/BG^{\frac{3}{2}}$ ($B = 85{\cdot}11$ measured) = weir coefficient in metric units.
$C' = 1{\cdot}811C$ = weir coefficient in British units.
$K \;= Q/(G + 32{\cdot}71)^{\frac{1}{2}}$ = coefficient of discharge of tunnel system for drowned conditions.

Fig. 11



Curves Showing Reservoir Level and Spillway Discharge for Four Alternative Arrangements of Supply Channel

Fig. 11 shows that curves for arrangements (*c*) and (*d*) relating to the shallow supply-channel of the prototype design give results intermediate between the two extremes—curves (*a*) and (*b*)—obtained with the deep experimental supply-channel. The effect of inserting a divide-wall in the optimum position is seen to be much greater for a deep channel than for a shallow channel. Tables 4 and 5 show that for the same gauge reading of 1·494 (slightly less than the designed value of 1·50) the effect of introducing a divide-wall into the deep channel is to increase the safe capacity from 231·6 to 316·4 cumecs, that is to say, by nearly 37 per cent. Conversely, the result of omitting the divide-wall would be disastrous; as shown in Table 4, a spillway discharge of only 241 cumecs would suffice to give a gauge-reading 1·294 metre in excess of the designed maximum value, and a slight further increase of discharge would inevitably result in the topping and destruction of the earthen dam. Moreover, even with a divide-wall, so much would depend upon its being placed in the best position that, in the present case, no recommendation for deepening the supply channel could be made without first ascertaining the best position for the divide-wall, by making further tests on a small model reproducing the exact topographical features of the prototype.

With a shallow channel such precautions appeared to be unnecessary. The curves of *Fig. 11* indicate that with the well-placed divide-wall of arrangement (*d*), the passage of a maximum discharge of 300 cumecs would be attended by a negligibly small excess of only 0·02 metre in the gauge-reading G; whilst if the divide-wall were omitted as in arrangement (*c*) the excess reading would be no greater than 0·26 metre. It appeared reasonable to infer from these results that a moderate error in locating the divide-wall would be unlikely to have any serious consequences.

In the light of the results obtained, it was therefore recommended that the supply channel should be excavated to the original design, and that a divide-wall should be placed across the channel on a line drawn through the centre of the spillway, normal to the contour-lines of the side-slope of the river valley.

WEIR COEFFICIENTS

In estimating the value of G as 1·5 metre for a discharge of 300 cumecs, the Chief Designer assumed a metric coefficient of 2·0, that is to say, 3·622 in British units. As is well known, in round-crested weirs the value of the coefficient, expressed as $C = Q/BG^{\frac{3}{2}}$, increases with discharge at a fairly rapid rate.

Except in the case of arrangement (*c*) in Table 6 where no readings were taken on low discharges, it is seen that the coefficient increases to the maximum and then decreases. For example, the decline in the values of C' in Tables 4, 5, and 7 is shown in Table 9 in British units.

In all cases, the normal tendency for the coefficient to increase with discharge is overcome by the progressively stronger effect of end-contraction due to oblique approach. As was to be expected, the decline is much greater in the absence of a divide-wall to kill the main uni-directional swirl in the supply channel. That a smaller decline also occurs when a divide-wall is inserted, is due to the fact that in this case it is only opposite the entrance, where the stream bifurcates, and in the vicinity of the divide-wall, that approach to the sub-weirs is radial. Elsewhere, and particularly near the entrance, there is still a strong tangential component of flow in the supply channel. It will be noted that, in spite of this effect, arrangement (*b*) with a deep channel still gives the average coefficient a higher value than that assumed by the Chief Designer; whereas, with the shallow channel of his design the coefficient falls to a value some 7½ per cent smaller than he assumed.

TABLE 9

Arrangement	Table	From	To
(*a*)	4	3·241 for $Q = 144{\cdot}1$	2·700 for $Q = 231{\cdot}6$
(*b*)	5	3·800 ,, ,, = 233·0	3·688 ,, ,, = 316·4
(*d*)	7	3·453 ,, ,, = 271·3	3·355 ,, ,, = 301·5

TUNNEL-SYSTEM COEFFICIENTS

Below are shown, in ascending order of magnitude, the values of the tunnel-system coefficient $K = Q/(G + 32{\cdot}71)^{\frac{1}{2}}$ obtained from observations taken when the weirs were drowned and the gauge reading G was dependent not upon the weir coefficient, but upon the working head $H = (G + 32{\cdot}71)$ on the tunnel-system required to pass the discharge observed. By way of comparison, values of the working head $H = (300/K)^2$ required to deliver a standard discharge of 300 cumecs, have been computed for the four different arrangements.

TABLE 10

Arrangement	Table	K	$H = (300/K)^2$	Supply channel	Divide wall	Vortex
(*a*)	4	40·47	55·0	Deep	Out	Single
(*c*)	6	51·95*	33·3*	Shallow	Out	,,
(*d*)	7	53·40	31·5	,,	In	Twin
(*b*)	5	53·80	31·1	Deep	In	,,

* In the case of arrangement (*c*) the weirs were not fully submerged. If they had been, K would have had a smaller value, and H a larger, than those shown above. With this reservation, the figures serve to show the great effect a single vortex has in reducing the potential capacity of the tunnel-system.

With twin vortices playing in the bellmouth, resultant spin and air indraft are slight and intermittent: with a single vortex (*Figs 5* and *6*), air indraft is copious and continuous, and a strong left-handed spiral flow persists along the greater part of the tunnel. Although air indraft plays a part by reducing the sectional area of the water-stream, it would appear that it is the spiral flow of the stream, rendering only the axial component of velocity effective, that is the main factor in reducing the value of the coefficient K.

Normal Flood Performance

The Chief Designer was anxious to obtain confirmation of his expectation that a normal flood of 150 cumecs would be passed under atmospheric (that is to say part-bore) conditions of flow. Actually, as already stated, the usual phenomenon of sudden breakdown with afflux did not occur in any of the tests. Instead of this, the bore of the tunnel was first filled by a muff of water enclosing a pocket of air which formed at an early stage, under complex conditions of flow, a little downstream of J1 where the elbow gave into the slow bend. When this took place, a small core of richly aerated water made its appearance in the throat of the bellmouth. As the discharge increased, the core rose quite steadily into the bellmouth without any abrupt changes. At the normal flood stage of 150 cumecs, the core stood at a steady level about 6 metres above J0 (the base of the shaft). Piezometer readings showed steady pressures: T1 a negative pressure of about 1·8 metre of water: T2 a negative pressure of 0·5 metre, and T3 virtually atmospheric pressure.

It was considered that the conditions observed offered no threat of any action likely to cause damage to the lining of the tunnel.

Piezometric Measurements

Measurements of pressure in the elbow were made by connecting manometers to the four piezometric tappings shown in *Figs 2*, p. 577.

For measuring vacuum pressures at tappings T1 to T3 a U-tube air-manometer was used, the tappings being connected in turn. For measuring positive pressures at T4 a water-manometer was permanently connected to this tapping. A second water-manometer was used for measuring positive pressures at T3 when the elbow was running full-bore at this point. Readings taken with the three alternative arrangements, (*a*), (*b*), and (*d*) are shown in Table 11.

The pressure-heads at piezometric tappings T1 to T4 are measured (*AT*) by U-tube air-manometer, or (*WT*) by water-manometer, with arrangements:

(*a*) deep supply-channel with no divide-wall;
(*b*) deep supply-channel with divide-wall in place; and
(*d*) shallow supply-channel with divide-wall in place.

TABLE 11

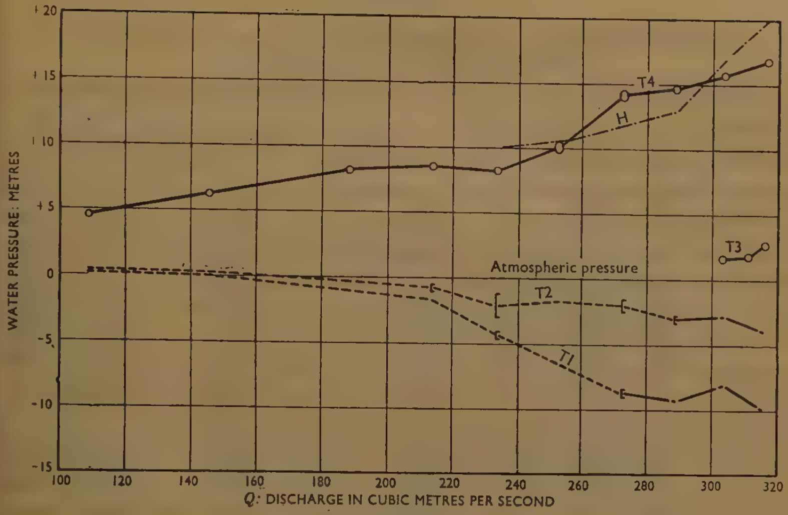
Arrangement	Discharge: cumecs	Pressures in prototype: metres of water					H	Remarks
		*AT*1	*AT*2	*AT*3	*WT*3	*WT*4		
(*a*)	107	− 0·74	+ 0·08	—	—	+ 5·78	—	Air pocket along roof. No core in bellmouth.
"	144	− 1·50	+ 0·25	—	—	+ 7·01	—	
"	194	− 4·60	− 1·10	—	—	+ 7·91	9·5	Core in bellmouth at height *H* above top of bend. Full-bore flow in tunnel.
"	211	− 3·60	− 1·50	—	—	+ 6·81	18·0	
"	232	− 3·35	− 2·00	—	—	+ 8·61	19·0	
(*b*)	109	+ 0·10	+ 0·15	—	—	+ 4·61	—	Air pocket along roof of bend: no core in bellmouth.
"	145	zero	+ 0·10	—	—	+ 6·31	—	
"	188	− 1·15	− 0·30	—	—	+ 8·21	—	
"	213	− 1·60	− 0·80	—	—	+ 8·68	—	
"	233	− 4·40	− 2·10	—	—	+ 8·36	10·0	Core in bellmouth at height *H* above top of bend. Air pocket in bend.
"	252	− 6·50	− 1·75	—	—	+ 10·11	10·5	
"	272	− 8·80	− 2·00	—	—	+ 14·21	12·0	
"	288	− 9·30	− 3·05	—	—	+ 14·81	13·0	Full-bore flow in tunnel and bend Core at height *H*.
"	303	− 8·10	− 2·80	+ 1·00	+ 1·71	+ 15·86	17·0	
"	311	—	—	—	+ 1·91	—	19·0	
"	316	− 10·00	− 4·00	—	+ 2·74	+ 16·91	20·0	
(*d*)	108	zero	+ 0·10	+ 0·30	—	+ 4·51	—	No core: air in pocket in bend. Incipient core.
"	142	− 1·20	− 0·20	+ 0·25	—	+ 6·20	5·0	
"	190	− 4·80	− 1·50	− 0·60	—	+ 7·11	9·0	Air pocket in bend intermittently isolated by blockage in horizontal bend.
"	233	− 10·70	− 1·00	+ 1·50	—	+ 10·64	14·0	
"	254	− 11·50	− 0·25	+ 2·00	—	+ 13·11	15·5	
"	272	− 8·00	− 4·20	—	—	—	16·0	With *Q* + 233 cumecs tunnel runs full-bore.
"	288	− 11·50	− 8·00	− 1·00	+ 0·34	+ 14·64	18·0	
"	303	− 9·20	− 3·00	zero	− 6·69	+ 13·91	19·0	

Prefixes *A* and *W* indicate the use of air- and water-manometers respectively. Results obtained for arrangements (*b*) and (*d*) only are plotted in *Figs 12.* These show discharge and water-column pressures in terms of the prototype. Taking a column of 10 metres of water as representing a pressure approaching to a perfect vacuum, it is seen that this condition is attained or exceeded only at tapping T1 near the upper end of the elbow.

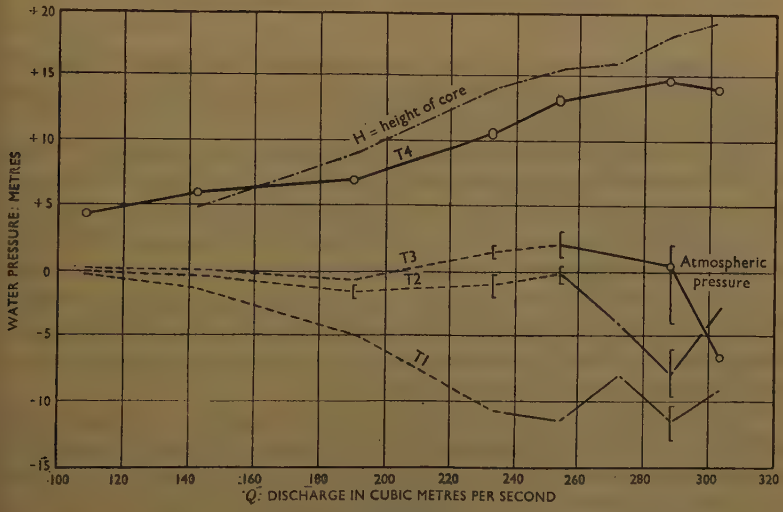
Table 12 represents an attempt to ascertain the cause of low pressures at T1.

Neglecting small frictional losses in the bellmouth, water falling from the surface of the core (*H* metres above this section) into a region at pressure (−*AT*1), would acquire the velocities shown in the Table. The

Figs 12



ARRANGEMENT (*b*)



ARRANGEMENT (*d*)

CURVES SHOWING PRESSURES IN ELBOW FOR DIFFERENT DISCHARGES

area $a = Q/V$, which may be designated, "the apparent effective area of the jet," is in all cases found to be substantially less than the full area of 16·60 square metres of the throat, as indicated by the tabulated values of $\frac{a}{16\cdot60}$.

TABLE 12

Calculation of velocity and sectional area of jet at top of vertical bend for arrangements (*a*), (*b*), and (*d*) respectively

Arrangement	Q	H	$-AT1$	$H + AT1$	$V = \sqrt{2g(H + AT1)}$	$a = \frac{Q}{V}$	$\frac{a}{16 \cdot 60}$
(*a*)	194	9·5	4·6	14·1	16·63	11·67	0·703
,,	211	18·0	3·6	21·6	20·60	10·24	0·617
,,	232	19·0	3·35	22·35	20·93	11·10	0·669
(*b*)	233	10·0	4·4	14·4	16·82	13·85	0·835
,,	252·5	10·5	6·5	17·0	18·27	13·82	0·832
,,	272	12·0	8·8	20·8	20·20	13·47	0·811
,,	288	13·0	9·3	22·3	20·92	13·77	0·830
,,	303	17·0	8·1	25·1	22·20	13·66	0·823
,,	316	20·0	10·0	30·0	24·26	13·03	0·785
(*d*)	233	14·0	10·7	24·7	22·00	10·58	0·636
,,	254	15·5	11·5	27·0	23·00	11·04	0·665
,,	288	18·0	11·5	29·5	24·05	11·97	0·721
,,	303	19·0	9·2	28.2	23·53	12·89	0·776

The values of Q, H, and $AT1$ taken from Table 11.

The presence of some air in the throat section at all stages of discharge is responsible for some reduction in area, but certainly not to the extent indicated in the Table.

The effect of spiral flow in the throat has also to be considered. Values of V as calculated must be regarded as resultant velocities: with spiral flow, the true effective area would be greater than that shown in the Table in the ratio of the secant of the angle of obliquity of the spiral. This would explain why arrangement (*a*) with its strong single vortex shows lower values of a than does arrangement (*b*) with its opposing twin vortices. The smaller maximum capacity of arrangement (*d*) compared with that of arrangement (*b*) is only to be explained by its higher vorticity, as is indicated by lower values of a in the Table.

The Author is not satisfied that the two reasons considered above suffice to explain the low values of $a/16 \cdot 60$, particularly those exhibited by arrangements (*b*) and (*d*). It is doubtful whether centrifugal action can take effect at a section so close to the beginning of the bend, and it seems possible that with the abnormally high velocities obtaining, the shape given to the bellmouth is not good enough to prevent the formation in the throat of a *vena contracta*.

The Paper is accompanied by eight photographs, and four sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared.

Discussion.

The Author introduced the Paper with the aid of a film and lantern slides.

Mr H. D. Morgan observed that some of the problems involved in the design of the Fassideri project had been such that they could be solved only by means of a model. An earthen dam, or any large dam which incorporated a spillway of the closed-tunnel type, whether bellmouth or otherwise, placed considerable responsibility on the designer, because a tunnel had a definite upper limit of discharge. An open spillway, whether it was one with gates or simply a weir, always had something in hand; the water level might rise a little higher than was desired, but there was always a little freeboard to spare, and the discharge was increasing very rapidly with the head, so that if the flood-discharge pipe were not quite right there was still a kind of safety valve. With a discharge tunnel, on the other hand, there was a definite upper limit.

He himself had had to design a dam quite recently for a tropical country with very heavy rainfall, and it had been necessary to use discharge tunnels. He had felt a little worried about it. After all, flood prediction was extremely difficult; it could be done only on a statistical basis and, as a rule, the necessary information was not available. Records were very frequently pitifully meagre, and a decision had to be made as to what would constitute a catastrophic flood. The design might be based on a 1,000-year periodic or a 500-year periodic, but that did not mean that the catastrophic flood might not occur the following year.

The dam of which he was speaking was a rockfill dam and had a road across the top. It had been designed so that if there were at the worst 18 inches or 2 feet of water over the road it could go down the bank without doing any harm.

He was familiar with the model described by the Author and found it extremely interesting to watch in operation. The most striking thing was the vortex action by which some of the available potential energy of head was wasted as kinetic energy of rotation, and the discharge therefore suffered. It would be recalled that Dr W. C. Unwin deduced a rational formula for the discharge of a flat-topped weir by assuming the flow-filaments to be sensibly parallel to the crest and the stream of depth t.

The discharge Q was then written down in terms of t and the assumption made that it would adjust itself until Q was a maximum. By differentiating and putting $\frac{dQ}{dt} = O$, a value of t was found for maximum Q. That gave $t = \frac{2}{3}H$ and the formula followed. In the case in question, however, a similar line of reasoning failed because of the vortex occurring. Even in open water, without any initial tendency to rotation—though that was present, of course, with the spillway—a vortex would form; the slightest eddy would tend to start a vortex in one direction or the other. The model clearly showed the astonishing amount of energy which could be lost by that action. It was possibly somewhat disappointing, having put in a dividing wall, to get a pair of vortices; it seemed to be a most persistent action, which was very difficult to kill.

On the subject of the basin at the bottom of the spillway, Mr Morgan observed that that also presented a problem which was not susceptible of analysis. The only way in which it could be dealt with was on a model. The model described by the Author had been extremely interesting, because one might spend months making drawings of all kinds of spoiling devices and still be no nearer to a definite solution, but one could get it on a model, and one had in the case in question almost perfect dynamic similarity.

Finally, Mr Morgan said that it was a great pleasure to see the Hydraulic Research Station getting under way. Great Britain had at one time lagged behind in such research. There were as good brains in Great Britain as elsewhere, but no-one had bothered about that subject before. He was very glad to see the present example of what could be done in an investigation of the kind in question.

Mr Gerald Lacey drew the Author's attention to a Paper by the late Mr W. J. E. Binnie,[1] a Past-President of the Institution. That Paper had been published a year before the earliest Paper quoted by the Author, and it would surely be a source of pleasure to members of the Institution that the first Paper on the subject had been presented by a member who afterwards became President. It was of great interest to note that one of the matters to which Mr Binnie particularly referred was the difficulty of the vortex. Mr Binnie's Paper dealt at considerable length with the influence of a curtain wall dividing the bellmouth into two semi-circular weirs. In other words, he had found exactly the same difficulty as the present Author and had dealt with it not by one divide-wall but effectively by two, or rather, by means of a single wall which went straight across the bellmouth, and followed the curve for a short distance down each side. Mr Binnie had not had the separate piers which were characteristic of the bellmouth of which the Author had spoken.

One matter which puzzled Mr Lacey a little was the question of the rugosity of the tunnel both in the prototype and in the model. The Chief Designer had felt nervous about the question of the correct value of Kutter's N to adopt. Kutter's equation was hardly applicable to pipes, and perhaps Manning's would be better; but if a figure such as 0·014 was quoted for Manning's equation for a pipe, before that could be interpreted it was necessary to know the discharge, the diameter, the Reynolds Number, and the temperature of the water; because until that information was available one did not know whether one was dealing with smooth or rough turbulent flow. With rough turbulent flow, Manning's rugosity coefficient implied a relative rugosity which was easy to understand, but if one were dealing with smooth turbulent flow and applied Manning's equation, Manning's N had nothing to do with the roughness, and had to be interpreted in terms of the Reynolds' Number of the fluid. Mr Lacey would like to know, therefore, whether in the case in question there had been full turbulence both in the model and in the prototype. He imagined that there must have been, but he was perturbed by the extraordinary difference between the Author's estimated losses down the chute, and the actual losses which had been encountered. The Author said that the flow was very smooth, but was it smooth turbulence or rough turbulence, or was the flow down the chute a particular type of flow about which very little was known? It was curious that the calculated losses for the prototype were about 2 metres, whilst those measured from the model were only about 0·2. That was very surprising, because if one did not get those losses in the prototype the standing wave would be pushed right out of the basin.

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[1] W. J. E. Binnie, "Bellmouthed Weirs and Tunnel Outlets for the Disposal of Flood Water." Trans Instn Water Engrs, vol. XLII (1937), p. 103.

Mr Lacey wondered if, should there be a real danger of that occurring in the prototype, there would be any objection to roughening the chute, and making some attempt to dissipate part of the energy on the chute itself. He had been concerned with computing the losses on the glacis upstream of the standing wave on the floor of a large barrage, and in view of the difference between the Author's calculations and the actual model results he would very much like to know how that could be done accurately. In the particular example, he thought that it would have been a definite advantage to roughen the chute.

Mr G. A. J. Young said that he had had some connexion with the Fassideri model project in its early days and also had recently been conducting model investigations on two similar bellmouth-spillway projects in the laboratory of the British Hydromechanics Research Association at Harlow in Essex. He would like to refer later to some interesting points which had arisen from those investigations, but first he wished to discuss two points which arose from the Paper.

On p. 581, the Author had compared the frictional losses in the model with those in the prototype. He had assumed values of Kutter's N of 0·009 for the perspex tunnel and of 0·0136 for the concrete tunnel of the prototype, and had argued that the frictional losses in the model were about 50 per cent higher than those in the prototype. On the strength of this argument, he had increased the slope of the model by about 50 per cent in order to ensure similar conditions of flow throughout the whole range of part-bore flow, and had also shortened the model tunnel by 34 per cent in order to ensure similarity in the region of full-bore flow.

Fig. 13 showed the familiar Moody diagram in which the friction factor f in the d'Arcy formula was plotted against the Reynolds' Number. The value of Kutter's N equal to 0·009 assumed for the model perspex pipe corresponded to a value of f in the d'Arcy formula equal to 0·0207, which was about 45 per cent higher than that for a smooth pipe at the model Reynolds' Number of 3×10^5, and corresponded to a relative roughness of 0·001, which meant an equivalent sand roughness of about 0·005 inch.

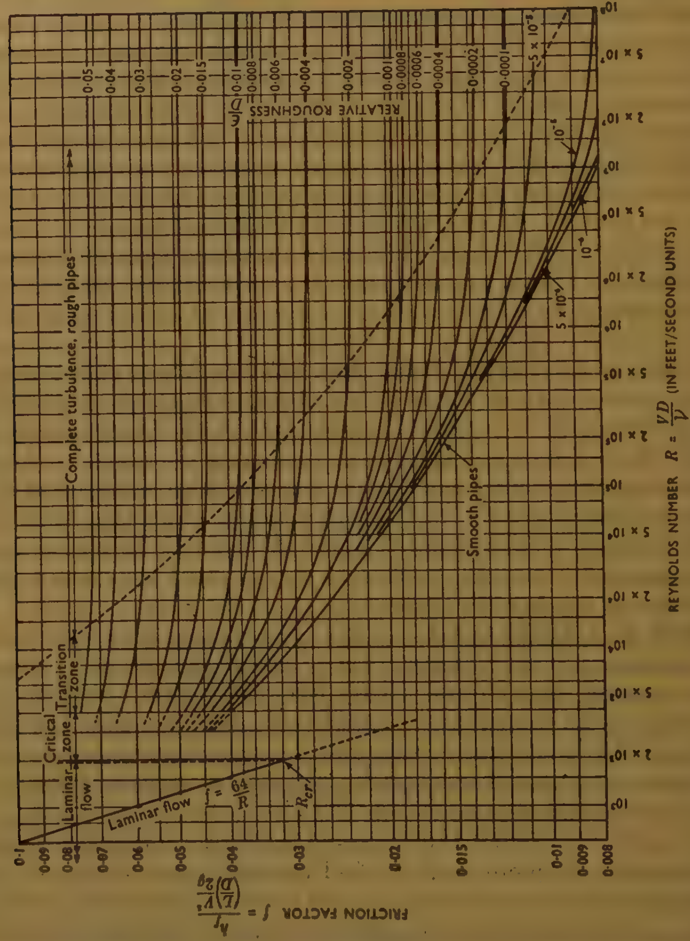


Fig. 13

Moody Diagram

The experiments which had been conducted at Harlow with a 6-inch diameter perspex pipe had shown that the perspex pipe might be regarded as hydraulically smooth, giving a friction factor f in the d'Arcy formula of 0·015 at a Reynolds' Number of $2{\cdot}6 \times 10^5$, which corresponded to a value of Kutter's N of 0·0075.

It would be interesting to know whether the Author had made any measurements of the frictional losses in his model perspex tunnel, and if so, whether they confirmed the rather high value of Kutter's N which he had assumed. If in fact the model tunnel might be regarded as smooth, then the value of f for the model was 0·0145, and the value of f for the prototype, corresponding to a Kutter's N of 0·014, was 0·0147. That would indicate that it was perhaps unnecessary to tilt the model tunnel and to shorten it.

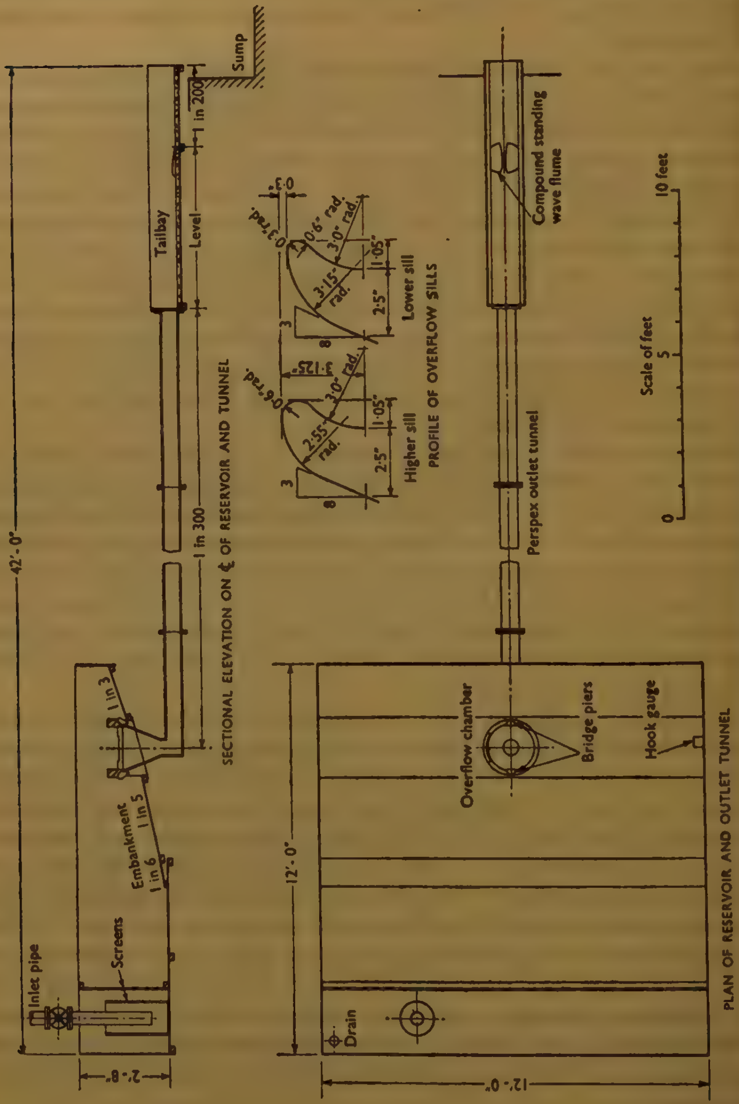
On p. 595 the Author had discussed the pressures measured along the elbow at the entrance to the tunnel. *Figs 12* indicated that the pressure on the inside of the elbow at the entrance would fall to a value approaching the vapour pressure of water, and that cavitation might occur. It should perhaps be pointed out that the model as it stood would not reproduce those cavitation phenomena, since the absolute pressures occurring in the model were relatively too high and did not approach the vapour pressure of the water. Those cavitation phenomena could be reproduced on the model either by reducing the air pressure above the free water surface, so that the absolute pressures were to scale relative to the water vapour pressure, or by raising the water vapour pressure by using boiling water. The former had been tried successfully, but Mr Young had not heard of any attempts at the latter.

Referring to the experiments at Harlow on a model of the spillway for the Weir Wood Reservoir scheme, Mr Young said that the scheme had been designed with a mitred elbow at the entrance to the outlet tunnel, as shown in *Figs 14.* At low discharges (less than 500 cusecs), when there was no core of water in the overflow chamber, the nappe left the bottom of the conical overflow chamber as a free annular jet and impinged on a local area of the tunnel floor. These jets had a high velocity, and it had been feared that their continued action over a small area of the floor might lead to undesirable wear.

In order to reduce that effect, various forms of deflector ring had been inserted near the bottom of the cone, and the arrangement finally adopted was shown in *Fig. 15.* The water flowing down the sides of the overflow chamber was deflected upwards by the chamfer on the ring and in towards the centre, where the jets from opposing sides collided and the water fell like heavy rain from a height corresponding to about 20 feet in the prototype, on to the floor. It had been felt that that arrangement would minimize the wear on the tunnel floor at the entrance to the tunnel. When the tunnel was running full bore and there was a core of water in the overflow chamber, the ring, with its nozzle-shaped entry, offered very little additional resistance and had a negligible effect on the maximum capacity of the system. The small radius provided at the entrance to the vertical portion of the tunnel offered no additional resistance which could be measured.

It would be appreciated that the maximum discharge which the system could safely carry was determined by the overall resistance of the bell-mouth, mitre bend, and outlet tunnel under full-bore conditions. In the particular scheme in question it had been found that the losses in the mitre elbow represented about 45 per cent of the total losses, and it had been

Figs 14

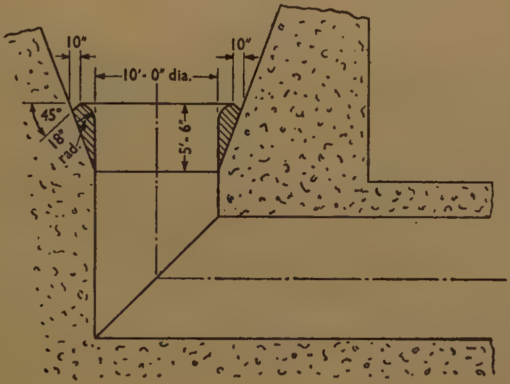


WEIR WOOD RESERVOIR SCHEME, 1/20TH-SCALE MODEL

estimated that the maximum capacity could be increased by about 2 per cent if the mitre elbow were replaced by a circular bend.

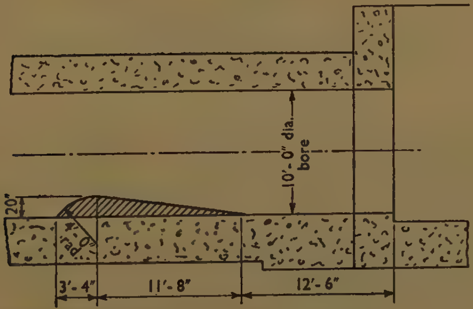
That had been tried on the model, but had been at first unsuccessful, because the outlet tunnel did not fill, even when the overflow chamber was full and the weir drowned. That appeared to be explained by the fact that the water flowed round the outside of the bend with a high velocity, whilst separation occurred at the inside of the bend, and the flow was hypercritical throughout the whole length of the tunnel. It had been found that the tunnel could be filled by creating a hydraulic jump near the end of the tunnel, which would cause the tunnel to run full-bore at outlet. The rest of the tunnel would then fill as the hydraulic jump progressed upstream and the entrapped air was removed by a process of entrainment of air bubbles in the rollers of the jump.

Fig. 15



Deflector Ring at Bottom of Overflow Chamber

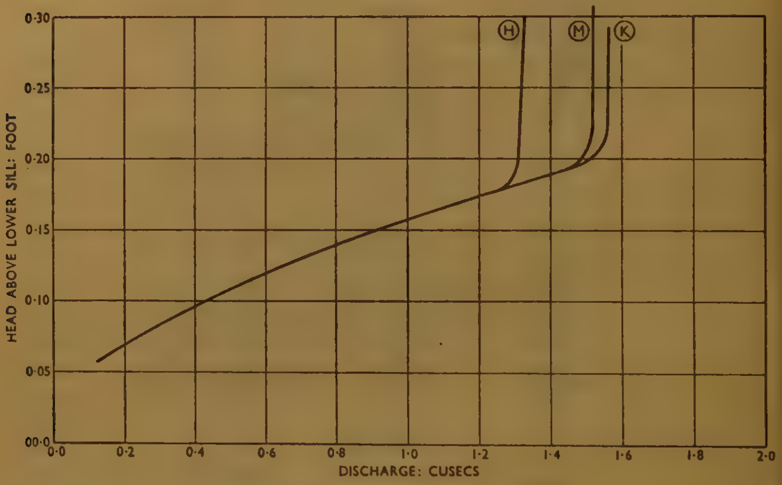
Fig. 16



Profile of Hump Inserted near Tunnel Outlet

In order to create a hydraulic jump at the end of the tunnel a hump was inserted just inside the end of the tunnel, as shown in *Fig. 16*. The effect of those modifications on the maximum capacity of the system was shown in *Fig. 17*. Curve H was the calibration obtained with the original mitre elbow. Curve K had been obtained with the circular bend at the entrance to the tunnel, when the tunnel was made to fill artificially by inserting a temporary obstruction which was then removed. The tunnel had then filled itself of its own accord. Curve M referred to the arrangement of the circular bend with the hump at the end of the tunnel.

Fig. 17



Calibration of Overflow Weir with Circular Bend at Entrance to Tunnel (Model-values)

The difference between curves M and K was explained by the additional resistance of the hump under the full-bore conditions.

Professor Herbert Addison, referring to the film which the Author had shown, asked whether any special technique had been involved and whether the film had been made by specialist film producers or by the staff of the research organization.

Had the experiments thrown any light on the question of the minimum ratio between the bellmouth diameter and the pipe diameter? The whole apparatus was really a large pipe for evacuating water from the reservoir. The spillway was a kind of enlarged bellmouth. It would be very expensive to construct, and the smaller the diameter of the spillway in relation to the size of the pipe the less expensive the whole construction would be. Had any information been gathered as to what was the optimum size, and could the spillway diameter possibly have been reduced, thereby reducing the cost of the complete structure?

Referring to the question of the relative friction factor in the prototype and in the model, he asked whether it was really completely permissible to treat the pipe as a normal long straight pipe. Before reaching the pipe the water had had a rough time: it had come over the spillway and been very much disturbed by the vortex, and had then come round a bend; by the time that it got into the straight pipe, was it in a condition to obey the normal laws which might be expected to apply to a straight pipe? Until that was definitely known, he was not sure that merely to take values from curves applying to long straight pipes would be completely admissible.

The Author, in reply, said that Mr Morgan had referred to the difficulty of predicting the magnitude of the maximum flood ever to be expected. With a device such as a bellmouth spillway or a siphonic spillway, liable to a rapid rise of surface which was certain to wreck an earthen dam, it was imperative to be on the safe side. M. Michalopoulos, the Chief Designer, just doubled for luck; his records led him to believe that he would never get anything more than 150 cumecs, and so he proceeded to design for 300. The Author thought that that was a pretty good guess and should be safe. On a statistical basis there was no limit, but to double for safety seemed to be a good rule.

It depended, of course, on the nature of the phenomenon, but he could give an example from his own experience. He had spent most of his life in India, and was for a long time on the Jhelum, the most northerly of the Punjab rivers. He had been posted there as a young man in 1907 during the construction of a new canal, when great pains had been taken to ascertain what the maximum discharge of the river would be at the site of the canal regulator. It was an unusual type of regulator for the Punjab —a structure about 100 feet high and designed for 60 feet of rise. That was because it was in the submontane region where the river was swift, and there was four or five times the rise which occurred in the plains. The regulator had been constructed on the assumption that 500,000 cusecs was the biggest discharge that the river could ever attain. The regulator had been designed for a 60-foot rise and 10 feet of freeboard, but in 1929 there had been a flood of 900,000 cusecs, just short of double the previous record. It topped the whole structure by 2½ feet and severely damaged both flanks, but by the grace of God it went down just in time to prevent the work being outflanked and flood-water entering a cutting about 100 feet in depth, when it would have devastated the whole countryside.

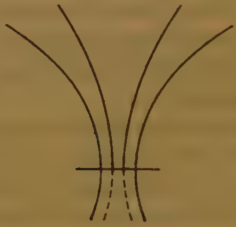
It was obviously impossible to lay down any hard and fast rules, but it was clearly imperative with a bellmouth spillway to make sure that it would do its job of protecting a very vulnerable structure such as an earthen dam, so that it must never be allowed to reach the stage of rapid rise. If doubling it would not do, one must treble or quadruple it. Engineers had been accustomed to factors of safety of 4, and, if uncertain, would even go to 10.

Mr Morgan had spoken of the principle of maximum discharge of a weir.

It might be of interest to mention that Professor A. M. Binnie had sugg gested a similar principle for a bellmouth in terms of the single vortex The bottleneck was at the throat, and one might cut away the lowe part (see *Fig. 18*) and consider the vortex discharging into air. Professo Binnie had worked out (for a given value of the head *H* and a given spin (expressed in terms of $C = VR$) the size of the core which gave the maximum value of discharge, and he had demonstrated by a number of observation that his theory held good.

The difficulty arose in measuring the spin. He had had to measure th piezometric head at the throat, but his results had vindicated the truth o the theory. The interesting point was that there was no limit to th reduction in discharge. Keeping the head fixed, if more spin was put int the vortex the water wall at the throat became thinner and thinner unti it was a mere film, so that there was no limit to the reduction of discharg that could be caused by spin. Keeping the head constant, one could ge any discharge one liked, from a bellmouth full of “ black ” water and n spin down to zero with infinite spin. In Professor Binnie's experiment he had set up an arrangement whereby he could admit the whole or an fraction of the supply peripherally, and the remainder radially. A compared with the 25-per-cent reduction obtained on the Fassideri mode and the 25–34 per cent that Mr G. M. Binnie had found in his models fo the Jubilee siphon, Professor Binnie, by imposing a particularly sever spin, got down to a discharge of one-thirteenth of the discharge whic would have been obtained under the same head with “ black ” water an no spin.

Fig. 18



The Author understood Mr Morgan to say that he distrusted theory i regard to the standing wave and that it would be necessary to use a mod as a guide in designing the stilling-basin. The Author did not entirel agree, because the theory of the standing wave was based on the momen tum principle. Viscosity did not come into it; it was purely a question momentum. He felt that it was one of the few cases where theory wou give a fairly reliable answer; worked out on the momentum principle gave a dimensionless relationship which was fairly correctly reproduced, whatever the scale.

He apologized for not having read W. J. E. Binnie's Paper. He had seen an excerpt from it, but had not had an opportunity of studying the whole Paper. He would make a point of doing so. He understood Mr G. M. Binnie's Paper purported to be an elaboration of those results, but he could not say that definitely, not having read the earlier Paper.

With regard to the relative rugosity of the prototype and the model, he expected to get the support of every engineer in regard to what the physicist might mistrust. He had assumed that engineers, with their practical methods of approach, could be fairly certain in assuming a value of N, and at the time he chose it he felt that he could not, for very shame, go lower than 0·009 for perspex. It had to be a guess. He had to admit, however, that he had not gone low enough. He had analysed some of Mr G. M. Binnie's results, using pipes of machined white-metal having perhaps about the same degree of smoothness as perspex, and he had worked back to see what roughness, expressed in terms of Manning, was required to account for the low heads with high discharges which Mr. G. M. Binnie had obtained under "black" water conditions. It called for an N for his pipes of about the order of that which Mr Young had mentioned, 0·0075. The Author admitted that his figure of 0·009 was too high; it overestimated the losses and, low value though it was, he thought that it ought to be considerably lower for the perspex. That meant, presumably, that he had not been justified in making the adjustments of slope and length which he had made, or that he had not made them in a correct manner. If he revised the value of N which he had taken for the perspex, he would have to revise that adjustment. Otherwise, he felt that the method adopted really involved two engineering guesses based on experience. A guess which would be safe for the N of a 15-foot-diameter circular concrete-lined tunnel would not be reliable for the smooth material. He had had support for his figure of 0·009 from Professor Allen, who, in his Table, gave 0·01.

With regard to Professor Addison's inquiry about the film, the work had been done by Mr Guthrie, the officer in charge of the Central Photographic Section at the National Physical Laboratory, and particularly by Mr Williams, who spent a great deal of time with a camera which possibly could have been very considerably improved upon, because it was only capable of taking a 1/32-second exposure; to do justice to models of the type in question the exposure should be much more rapid. The film could also, no doubt, have been greatly improved by charging the water with aluminium powder.

Mr Gerald Lacey had expressed some doubt with regard to the sort of losses likely to be obtained on a steep slope like the face of a high dam or a chute. The use of specially rapid photography had revealed that very peculiar things happened. A series of what promised to be very interesting articles had just commenced in *La Houille Blanche* on the subject of the theoretical approach and practical investigation into the behaviour of water descending a chute. The Author had seen photographs which suggested what happened. They were amazingly rapid, so rapid that they revealed what the human eye could not possibly detect, namely, that at a certain distance down from the crest, where the boundary layer had expanded to fill the whole section of the stream and come right to the surface, the water was simply a mass of little curly eddies looking almost like gravel. The water was apparently disintegrating. It just broke up and atomized into "white" water. How that affected the frictional losses he could not say, but that was what appeared to happen. Many people would have observed what happened in the case of a waterfall. There was a fall of rather low capacity in Natal with a sheer drop of 365 feet, and, depending on how much water was passing over, there was complete atomization in a free falling jet, just like water from a tap. There was a solid nappe descending a certain distance, and then the whole thing broke up, without being in contact with any boundary, and simply disintegrated. He thought that some very valuable information would probably be obtained from the series of articles which had been started in *La Houille Blanche*, and it might reveal that the losses were entirely different from those which would be obtained by normal methods of calculation.

Professor Addison had asked whether the tests had established any relationship between the diameter of the tunnel and the diameter of the spillway. The two diameters were entirely independent, and that of the spillway could be made as broad as one liked. The tunnel-cum-bellmouth system could not be allowed to take charge; the water could never be allowed to gorge the bellmouth. It was simply a problem, therefore, of ascertaining the discharge over a weir of given length. If one wanted to reduce the head one would simply adopt a larger diameter; on the othe hand, if one could afford a greater depth over the weir one could economize on the diameter of the mouth. It had very little connexion with the designing of the rest of the tunnel system, because in any case, whethe one built to one width or another, one designed for "black" water conditions, and the loss of head in the bellmouth was almost negligible compared with the losses in the rest of the system. It was not possible, therefore, to establish any connexion by experiment between tunnel diamete and the diameter of the mouth, which was at the option of the designer depending on what depth over the spillway he could afford.

Professor Addison had raised a very interesting point when asking ho an emulsion behaved and what would be the frictional loss. The onl possible way to design a bellmouth spillway or a siphonic spillway was however, on the basis of "black" water. That was an ideal conditio which could never be attained, but it represented the limit within whic it was necessary to keep. It was not until discharges became smaller tha that ideal that emulsification, low-pressure pockets, discontinuous pocket of air, etc., began to appear. It was of course conceivable that there coul be obtained such a high degree of emulsification that it would require a bigger head to pass the less dense material than would be required for " black " water, but he did not think that that was probable.

All sorts of very peculiar things happened, and he did not think that a model could possibly tell the whole truth. A model could not be expected to give a correct picture with regard to the proportion of admixture between air and water at different stages of discharge, or the way in which an emulsion would behave. He had seen a number of fairly large spillway siphons working, and they never worked with " black " water—always with an emulsion. So far as his observation went, he would say that, under similar conditions, the prototype was going to hang on to its air relatively very much longer than the model. In fact, what had surprised him in the model was that the air had come to the surface so quickly. He had expected to see a frothy mixture issuing at the exit of the tunnel, but in no circumstances in the model had the effluent been other than " black " water. On the prototype, he would be very surprised if for the most part the effluent was not just a seething mass. In other words, he thought that the water in the prototype would behave more like lemonade, and in the model like soda water, with violent " pops." Similarity could not be expected and it would be necessary to go to the prototype to get an answer to problems of that kind. It would be extremely interesting to know how, for example, the Jubilee siphon with its 250 feet of head had behaved, and to know whether the water actually issued, as the model would indicate, in a high-velocity jet with very little contained air, or whether it issued in a mad riot of foam and froth and " white " water, which personally he would expect, having observed large-scale siphons at work and large bodies of water in natural channels.

Correspondence on the foregoing Paper is now closed and no further contributions may be accepted.—Sec. I.C.E.

AIRPORT ENGINEERING DIVISION MEETING

24 April, 1952

Sir HUBERT WALKER, C.B.E., M.I.C.E., Chairman of the Division,
in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author

Airport Paper No. 20

"Dowel-Bar Joints for Airfield Pavements" *

by

John Allister Loe, B.Sc.(Eng.), A.M.I.C.E.

SYNOPSIS

It is not general practice in Great Britain to use dowel-bar joints in airfield concrete pavements although they are often used in highway construction and in airfield pavements abroad. Experimental and theoretical evidence and experience have indicated that the corners and edges are the weakest parts of concrete slabs and that some form of strengthening is desirable. Various methods of strengthening the edges have been devised but the use of dowel-bars is the most satisfactory when all aspects are considered and practice regarding their use is reviewed.

The performance of a dowel-bar joint in a concrete pavement is considered by examining the behaviour of a single dowel-bar under laboratory tests and then by applying the results obtained to an actual joint between slabs. The laboratory tests have provided data on the size and spacing of bars necessary to provide adequate strengthening of the slab, and this is tabulated for various conditions.

Changes in air temperature, by producing temperature gradients throughout the depth of the slabs, cause warping, and at night a slightly concave surface will be produced. This will result in increased vertical movements at the joints during the passage of loads; measurements have shown that dowel-bars are particularly effective for such conditions in reducing the stresses developed in the concrete by traffic.

The importance of careful construction of dowel-bar joints is emphasized and the effect of errors in alignment of the dowel-bars is indicated.

INTRODUCTION

IN Great Britain it is not usual to provide load-transfer devices at joint in concrete runways—a condition probably determined more by the dictate of economy and expediency than by any fundamental assessment of thei possible advantages. In the construction of concrete roads, however dowel-bars have been used for this purpose for many years and a good dea of information has been obtained on their efficiency and performance. Th information cannot necessarily be applied directly to the case of runway since there are many different factors involved, and it is the purpose of thi

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Paper to try to relate the results of road experience to runway practice. It is inevitable, therefore, that considerable reference will be made to road conditions.

Where a large area is to be paved with concrete it is usual to divide the area into bays or slabs to allow the concrete to expand, contract, and warp, and also to facilitate construction. If the joint between two slabs of given uniform thickness consists merely of a vertical break in the concrete, the joint will be structurally weaker than the rest of the slab and this will determine the load-carrying capacity of the pavement. If the slab edges can be strengthened, or if part of the forces produced by a load at the edge of one slab can be transmitted across the joint to the adjacent slab, a better-balanced and more economical design of pavement will be obtained.

The Paper will endeavour to show that, for many conditions, such a balanced design may be obtained by the use of dowel-bar joints. In addition, the advantages of such joints are that the effect of impact is reduced (see *Fig. 1*), the slab edges will be maintained in correct alignment should settlement of the pavement occur, mud-pumping may in some circumstances be delayed or prevented, and the dowels may also help to lessen rocking of the slabs if the edges and corners warp upwards owing to temperature gradients or other causes. The main disadvantages of such joints lie in the increased amount of care and work involved in the construction of the pavement (particularly when the concrete is being mechanically spread, compacted, and finished by a continuous process) and, possibly, in the higher cost of materials and installation. A pavement for carrying present-day heavy-wheel loads and for future wheel-loads of unpredictable size has to be of considerable thickness and, if the concrete slabs rest upon a very rigid base, dowel-bars will be less effective in strengthening the joints than if the slabs rest upon a less rigid base or directly on the subgrade. Dowel-bar joints may therefore be of greater value for small airfields or for those which have to be built on poor soils, than for those designed for very heavy loads and constructed on a good subgrade with a thick base. There is no unanimity of opinion as to whether a rigid or flexible base is preferable beneath concrete runway slabs. On the one hand it is contended that a rigid base—such as lean concrete—will limit the deflexions of the edges and corners of the slab under load, and that even when the slab is warped upwards, cracking can be prevented by proper design; in this case the presence of load-transfer devices will serve no useful purpose. On the other hand there is some evidence from road work that concrete slabs are more prone to cracking when laid on a rigid base than when on a more flexible base, and this view has been supported by at least one airport engineer.[1] If a more flexible base is to be preferred, it may be that a concrete slab with adequately dowelled joints and on a well-compacted granular base will prove to be a better form of construction for very heavy loads than slabs with undowelled joints laid on a rigid base.

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[1] The references are given on p. 649.

The use of steel dowel-bars on runways will have some influence on blind-landing control systems but it is understood that this may be corrected by a simple amendment to the calibration of the system.

By common usage the terms "dowel-bar" and "tie-bar" are generally understood by road and airport engineers to have the following meanings :—

> A *dowel-bar* is a bar, generally of mild steel, which has flexural rigidity and is used to transfer forces (in a vertical direction) across the joint gap. It is arranged to slide so that no horizontal forces are transmitted across the joint. The term "dowel-bar" was originally used when solid round steel bars were the only type used, but for convenience it is used in this Paper to include all shapes of section, both solid and tubular.
>
> A *tie-bar* is a bar whose main function is to prevent opening of the joint and in which the transference of vertical forces is a secondary consideration. Both ends of the tie-bar are bonded into the concrete and it may be desirable to increase this bond by using hooked or deformed bars. Tie-bars may be of sufficient size to transmit vertical loads or they may be used to hold in close contact the two faces of an interlocking joint, thus rendering it an efficient load-transfer device.

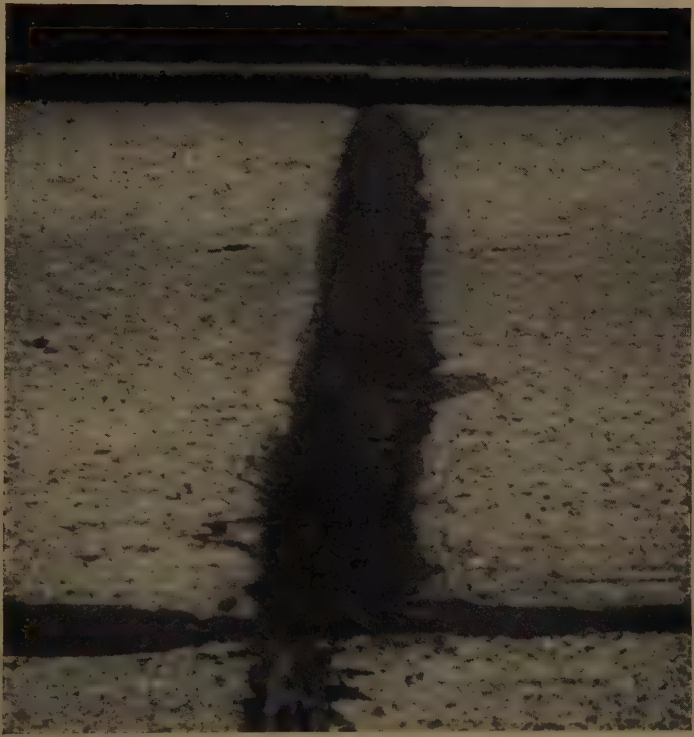
THE NEED FOR STRENGTHENING THE EDGES OF A CONCRETE PAVING SLAB

The need for strengthening the edges and corners of concrete slabs has been established from practical experience, experiment, and theoretical study. Weakness of the edges of the slabs may produce corner cracks which often develop in the manner shown in *Fig. 2*, or "D" cracks, so called because of the large "D"-shaped piece of concrete which is broken from the edge of the slab. It is probable that most corner cracks occur at night when the edges and corners may tend to be warped upwards; an example of such a crack occurring during the night, under a load which was satisfactorily carried during the day, has been reported by Snow.[2]

Experiments have also shown that the edges and corners are considerably weaker than the "interior" of the slab—that is, any part of the slab away from the edges and corners. Sparkes [3] has reported that the maximum surface strains resulting from the loading of a concrete slab 11 inches thick in an unwarped condition are as follows: for an 8-ton load at the interior: $37 \cdot 5 \times 10^{-6}$ inch; for a 4-ton load at an edge: $20 \cdot 5 \times 10^{-6}$ inch; for a 4-ton load at a corner: 54×10^{-6} inch. Skinner [4] reports experiments in which loads were applied through an 18-inch-diameter plate to an undowelled concrete slab 6 inches thick and the deflexions measured; the results, given in the following Table, have been taken from load/deflexion graphs.

As part of the investigation into the performance of concrete overlay

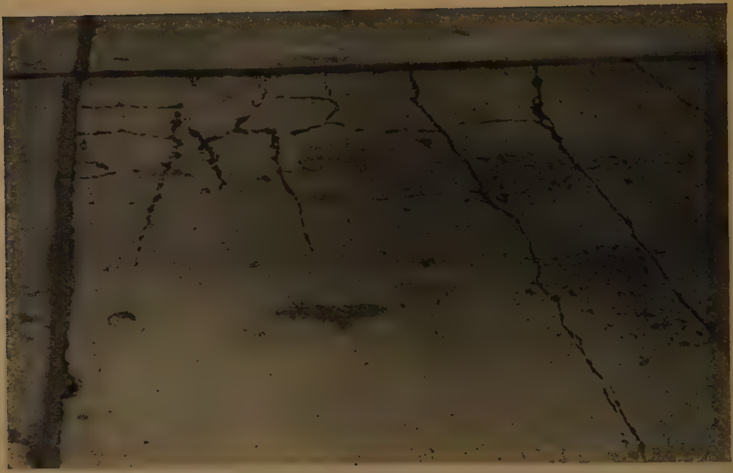
Fig. 1



SPALLING AT A JOINT HAVING NO LOAD-TRANSFER DEVICE

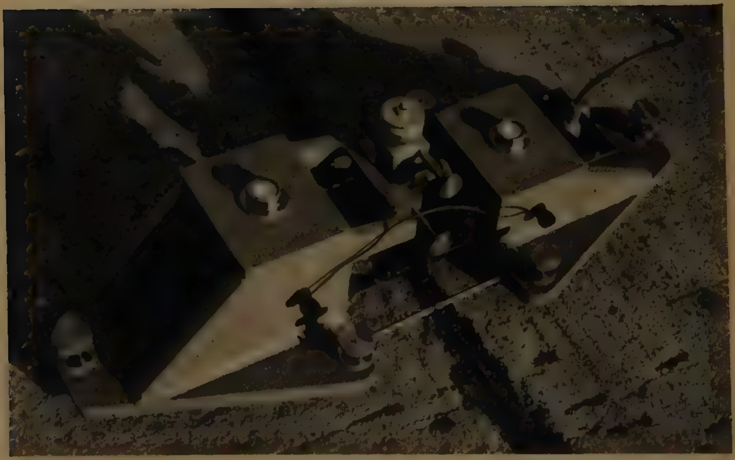
(The spalling is on the far side of the joint from the direction of approaching traffic and was probably caused by impact of wheels hitting the unloaded (and therefore undeflected) edge as they approached from the deflected adjacent slab.)

Fig. 2



Typical Example of Corner Cracking, showing Additional Cracks developing from the Original Break

Fig. 13



Electronic Apparatus for the Dynamic Measurement of the Vertical Movements of Concrete Slabs

Position of load	Subgrade reaction: 400 lb./sq. in./in.		Subgrade reaction: 150 lb./sq. in./in.	
	Load to produce a deflexion of 0·05 inch: lb.	Cracking load: lb.	Load to produce a deflexion of 0·05 inch: lb.	Cracking load: lb.
Interior .	50,000	70,000 plus	38,000	68,000
Corner .	13,000	20,000	8,000	13,000

pavements, the United States Army Corps of Engineers[5] carried out static loading tests on 25-foot-square panels of concrete of 12-inch thickness and resting on a 15-inch rock base. The deflexion produced by a load of 100,000 lb. applied at the corner was 0·40 inch, for the same load at the edge 0·24 inch, and at the centre 0·17 inch.

The theoretical study, made by Westergaard,[6] of the stresses produced by static loads on various parts of a concrete slab, shows that the stresses at the edges and corners are considerably greater than those at the interior. For example, for a load of 50,000 lb. applied through a tire having an elliptical contact area of 20 inches by 40 inches, the maximum stresses in a slab 10 inches thick laid on a subgrade having a modulus of subgrade reaction of 200 lb. per square inch per inch will be 445 lb. per square inch at the centre of a bay, 740 lb. per square inch at a longitudinal joint, and 665 lb. per square inch at a transverse joint. Teller and Sutherland,[7] who, as a result of experiments, made some modifications to Westergaard's original formulae, have shown that for a load of 10,000 lb. on slabs 8 inches thick, the following stresses will be produced :—

centre	160 lb. per square inch
edge, full subgrade support . .	305 ,, ,, ,,
deficient subgrade support .	340 ,, ,, ,,
corner, full subgrade support . .	260 ,, ,, ,,
deficient subgrade support	375 ,, ,, ,,

These figures suggest that if the stresses induced at corners and edges of slabs approximate to the ultimate strength in those parts, considerable relief of stress will be effected by the provision of a load-transfer device. If an efficient device is used the stresses at the edge may be reduced to approximately the same as those at the interior.

Methods of Strengthening the Edges of a Concrete Paving Slab

There are several methods by which additional support or strength may be given to the edges and corners of concrete slabs. Such methods may be classified broadly into three groups as follows :—

(1) Methods of strengthening the edges and corners.

(2) Methods of giving additional support to the edges and corners.
(3) Load-transfer devices.

In the first group, one of the earliest methods of strengthening the edges of a slab was by localized thickening of the slab. This method is not satisfactory because first, the subgrade or base has to be specially shaped; secondly, the thickened edges interfere with free expansion and contraction; and thirdly, while such an arrangement produces a better balance of loading stresses it may actually increase the combined stress due to loading and temperature gradients when the length of the slab is such that temperature warping is restrained.[8] Reinforcement, bent to the form of a hair-pin, has been successfully used to strengthen the corners and occasionally extra edge reinforcement has been used, but neither reinforcement nor thickened-edge construction maintains the slabs in horizontal alignment during loading or if non-uniform settlement occurs.

In the second group, the additional support may be given to the slab edges by means of concrete sleepers, but this method has the serious disadvantage that uneven settlement between the subgrade and sleeper may itself start cracks in the slabs. Another method which has been used recently is to construct the slabs in two separate layers of equal thickness (using the same quality of concrete), but staggered so that the joints in one layer coincide with the centre-lines of the slabs in the other layer—a thin dividing membrane separating the two layers. Further information about the performance of such slabs is awaited with interest but construction in two layers must involve increased thicknesses of concrete and more complicated construction procedures.

The third group consists of devices and arrangements by which part of the vertical force of a load on the edge of one slab may be transmitted to the adjacent slab. Many designs for such devices have been produced but of these the simplest, most effective, and most commonly used are dowel-bars for joints liable to open and close appreciably, and interlocking arrangements (moulded concrete projections or dummy joints) for transverse contraction or longitudinal construction joints where the amount of opening of the joint is likely to be very small.

Dowel-bar joints, if properly designed and installed, satisfy all the main requirements for load-transfer devices (see later paragraphs), but interlocking joints, whether consisting of interlocking projections moulded in the concrete or the interlocking faces of the cracked portion of a dummy joint, are most effective if the joint faces are in close contact. Opening of the joint greatly reduces the effectiveness of load-transference and may even render it useless in this respect, so it is important that these types of joint be used only in positions where the opening is likely to be small.

The quantity of dowel-bars required for a concrete pavement may be reduced considerably by using a combination of dowelled expansion joints and intermediate contraction, construction, or warping joints. If the intermediate joints are of the plain butt type with vertical faces, dowel-bars will be required to provide for load-transference, but since the joints will not usually open as wide as expansion joints, the dowel-bars may be of smaller diameter. If the intermediate joints are of the interlocking type, they should be arranged so that the amount of opening is small; a system commonly used for unreinforced-concrete roads is to space the expansion joints between 60 and 120 feet apart and construct intermediate dummy joints (or plane-of-weakness joints) at 15-foot intervals. There will be a tendency for the dummy joints to open progressively with a consequent closing of the expansion joints but, provided the amount of space for expansion is not excessive, the amount of opening of the dummy joints is likely to be small. Measurements made on an experimental road where the average expansion-joint spacing is 65 feet and the average dummy-joint spacing is about 20 feet have shown that the average amount of opening of the dummy joints during a period of 3 years was 0·02 inch. Openings of no greater magnitude than this are unlikely to render the interlocking aggregate particles ineffective in transferring load (Sutherland[9] states that dummy joints are not dependable when open more than 0·037 inch), and such joints may be constructed without dowel-bars unless it is desired to take extra precautions against the opening of the joint.

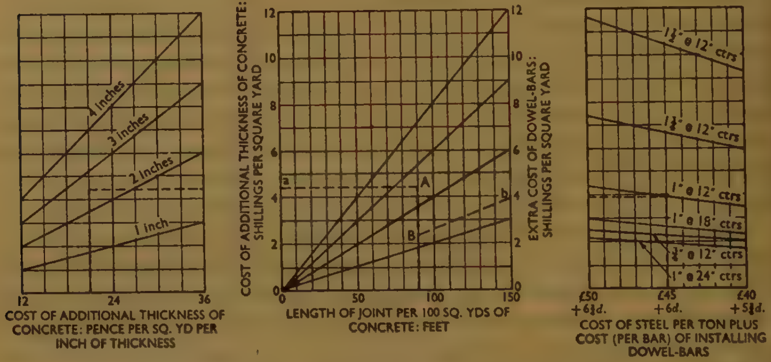
Warping joints are sometimes used in place of contraction joints; in such joints the two faces are held together by tie-bars, thus permitting the slab to warp without allowing horizontal movement. This type of joint is very effective in transferring load since the joint faces are kept in close contact; it also has the advantage that only a small quantity of steel is needed.

Longitudinal joints may be arranged in a manner similar to that suggested above, except that, in order to prevent opening of the longitudinal joints near the outer edge of the paved area, it is advisable to include tie-bars in these joints even if they are not used elsewhere.

The installation of dowel-bars will add to the cost of construction and it has to be considered whether it is more economical to use them or to use thicker slabs throughout. Which method is more economical will depend upon the spacing of the joints: with closely spaced joints the advantage is in favour of thicker slabs, but with more widely spaced joints it is in favour of dowel-bars. The theoretical costs of the above alternatives are compared graphically in *Figs 3*; the left-hand graph shows the cost of an additional thickness of concrete in shillings per square yard for various costs of concrete given in pence per square yard per inch of thickness. The right-hand graph shows the cost of dowel-bars of several sizes and placed at three spacings, for various costs of steel, and includes a charge for installation; this cost is given in shillings per square yard of concrete for a case where the length of joint amounts to 150 feet per 100 square yards of concrete (that is, for 12-foot-square slabs). Where the joints are spaced more widely apart, the length of joint per 100 square yards of concrete is less than 150 feet and the cost of the dowel-bars will be proportionately lower and may be read off the sloping lines in the centre graph.

An example will show how the comparison may be made: it is assumed that a slab is required to carry a wheel load of 25,000 lb. and that this requires an interior thickness of 10 inches. Joints are to divide the area into 20-foot squares. The alternatives are to use undowelled joints and increase the thickness throughout by about 25 per cent[10] (about 2½ inches at a cost of 21*d.* per square yard per inch of thickness), or to use a 10-inch thickness throughout but with joints having 1-inch-diameter dowels at 12-inch centres, at a cost of £45 per ton for the steel plus 6*d.* per dowel for preparing and assembling. (Lengths of dowel-bars are as in Table 1.) The extra cost of the thickness of concrete is denoted by point "a" (or "A") and the cost of the dowel-bars by point "b" (for 150 feet of joint per 100 square yards) or by point "B" for 90 feet of joint per 100 square yards of concrete (20-foot-square slabs) as in the example. The cost of the dowels is thus 2·3 shillings per square yard and compares with 4·3 shillings per square yard for additional concrete. The effect of dowel-bars on maintenance should also be taken into account, and the reduced wear on the joint filling and sealing materials, lessened risk of pumping, and the safeguard against "faulting" at the joints are other factors to be considered. If any of these faults occur, the cost of correcting them may considerably exceed the initial cost of the dowel-bars.

Figs 3



Example: Point A gives the cost of thickening a 10-inch slab by 25 per cent. (2½ inches) where the cost of the extra concrete is 21*d.* per square yard per inch thickness.

Point B gives the cost of strengthening the edges and corners by dowel-bars instead of by extra thickness. One-inch-diameter bars at 12-inch centres would be required and for 20-foot square slabs there is 90 feet of joint per 100 square yards of slab.

THEORETICAL COMPARISON BETWEEN COST OF ADDITIONAL THICKNESS OF CONCRETE AND COST OF DOWN-BARS

PRACTICE REGARDING THE USE OF LOAD-TRANSFER DEVICES

Practice regarding the use of dowel-bars varies considerably, but their use is recommended or specified by a number of authorities, particularly in the United States of America. The Civil Aeronautics Administration[10], in their manual on Airport Paving, state that provision for load-transfer should be made at all transverse expansion and construction joints and that dowel-bars should also be installed in contraction and dummy joints where the expected joint opening will be sufficient to destroy the interlock of the fractured surface of the crack below the groove. Where the runway is designed for a wheel load of 60,000 lb. or more, dowel-bars should be used in all joints. Pringle[11], in discussing the practice of the U.S. Army Corps of Engineers in constructing pavements for heavy wheel loads, states that dowel-bars are required at all transverse expansion and construction joints and are usually preferred for longitudinal construction joints. The sizes and spacings to be used vary with the thickness of the concrete slab but are the same for all types of subgrade and, for solid round dowels, are as follows :—

Thickness of slab : inches	Diameter of dowel-bar : inches	Minimum length of dowel-bar : inches	Spacing of dowel-bar : inches
Under 8	¾	16	12
8–11	1	16	12
12–15	1¼	20	15
16–20	1¾	20	18
Over 20	2¼	24	18

The practice of using dowel-bars is based upon observations of the performance of rigid pavements, and on investigations conducted by the Corps of Engineers' Rigid Pavement Laboratory. Dowel-bars of 2½-inch-diameter, 24 inches long and spaced 18 inches apart, have recently been used in a pavement 21 to 25 inches thick, designed for 75-ton wheel-loads.[12]

Accelerated traffic tests on airfields have generally shown that there is an advantage in using dowel-bars. Conclusions reached as a result of such tests on Hamilton Field [5] are that " dowels were highly effective in transmitting shear at the dummy and construction joints, even though the construction-joint dowels were sleeved. Dowels 1 inch in diameter and 16 inches long at 12-inch centres (the only size and spacing employed) safely carried moving loads of 280,000 lb. when the thickness of the slab was not less than 12 inches. Dowelled expansion joints were somewhat less efficient than dowelled dummy or construction joints. All of the dowelled joints were superior to the one type of unreinforced keyed joint employed between thin plain-concrete slabs in these tests. It would appear desirable to omit sleeves from construction-joint dowels in order to secure more effective action of this type of joint." Accelerated traffic tests on the Columbus Field[13] led to the conclusion that "dowel-bars provide an efficient means of transferring loads across the joints and in retarding pavement distress."

A different opinion is expressed by Philippe[14] who states that "in general the performance of a pavement is unaffected by the design of transfer devices except for keyed joints, which are generally unsatisfactory. Also, an adequate pavement is not appreciably weakened by weaker devices, and an inadequate pavement is not strengthened by stronger devices."

In Great Britain and Eire dowel-bars have been used only occasionally for airfield pavements; two instances of their use are (1) Hatfield airfield[2], where the expansion joints are provided with 1¼-inch-diameter dowel-bars, 24 inches long and spaced at 12-inch centres, and (2) Dublin Airport[15], where ⅞-inch-diameter dowel-bars, 21 inches long and spaced at 12-inch centres, have been used.

Considerable use is made of dowel-bars in the construction of concrete roads, and for trunk roads in Great Britain, the Ministry of Transport specify ¾-inch-diameter dowel-bars, 24 inches long and spaced at 12-inch centres, or a dowel plate, ½ inch thick and 4 inches wide, extending completely across the slab. Van Breemen and Finney[16], in co-operation with the members of an American Concrete Institute committee on the structural design of joints, survey the present practice regarding joints in concrete roads. They state that "It has been sufficiently demonstrated, and now generally recognized, that under heavy truck traffic some type of mechanical load-transfer device is essential to the preservation of the structural integrity of the pavement, and to the maintenance of desirable riding qualities. To accomplish this there is a definite trend toward the use of larger diameter dowels at closer spacings. The ¾-inch dowel is rapidly being replaced with ⅞- to 1¼-inch dowels spaced at 12- to 15-inch intervals. Dowel length still varies considerably. Research indicates that 15 to 18 inches is satisfactory. Experience indicates that on heavy trucking routes, the larger dowels are necessary." Van Breemen[17] has also stated that "in New Jersey the solution to the problem of designing pavements that will resist the damaging effects of heavy truck traffic has not been sought through improvements in subgrades alone. On the contrary as a means of strengthening the pavement itself, it is the current practice to install sturdy load-transfer devices at all joints—there being ample evidence that under heavy truck traffic, and despite all reasonable improvements in subgrade, it is absolutely necessary to adequately dowel all joints, otherwise an excessive amount of faulting will sooner or later occur."

THE REQUIREMENTS OF A DOWEL-BAR JOINT

The requirements of a dowel-bar joint may now be considered in detail, and may be stated to be principally as follows:

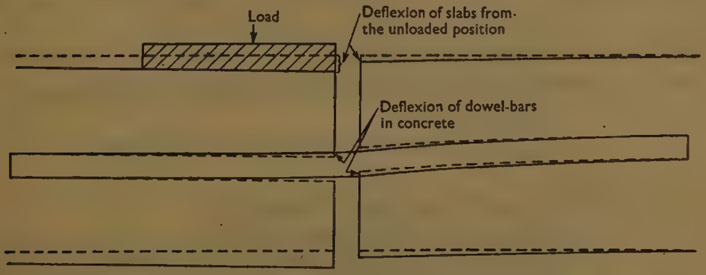
(1) The joint shall effectively transfer load between the slabs (the effectiveness of the dowel-bars in reducing stress in the slab, in preserving alignment of the slabs during loading, and in preventing excessive movement in the joint filling and sealing materials will depend upon this factor).
(2) The dowel-bars shall not restrain the freedom of the slabs to expand and contact.
(3) The dowel-bars shall not unduly restrict the freedom of the slabs to warp.
(4) The forces shall be transmitted without risk of failure (either by bending of the dowel or by crushing of the concrete).
(5) The dowel-bars shall have adequate strength to preserve the alignment of the slabs at the joints should any non-uniform settlement of the subgrade occur.

The effectiveness of dowel-bars in transferring load will now be discussed in detail.

THE BEHAVIOUR OF A SINGLE DOWEL-BAR: THEORETICAL STUDY AND LABORATORY TESTS

It has already been noted that, of the various methods of strengthening the edges of a slab, dowel-bars are generally preferable. The behaviour of the dowel-bars in a joint will now be considered, dealing first with the case of a single dowel-bar. When one side of the joint is loaded the dowel-bars will deform in the manner shown in *Fig. 4*. It will be seen that the overall deflexion consists partly of the deflexion of the bars across the joint gap and partly of the deflexion of the bars in the concrete. The maximum bending moment in the bars will occur in the two planes formed by the vertical edges of the concrete, the maximum shearing force will be constant across the width of the joint gap, and the maximum pressure between the bar and concrete will be at, or close to, the edge of the concrete slab.

Fig. 4


Load
Deflexion of slabs from the unloaded position
Deflexion of dowel-bars in concrete

SECTION THROUGH A DOWEL-BAR JOINT LOADED ON ONE SIDE

The effectiveness of the joint, having dowel-bars placed at mid-depth, will depend upon :—

(1) the size and shape of the bars,
(2) the spacing of the bars,
(3) the width of the joint,
(4) the method of preventing bond between bar and concrete,
(5) the strength of the concrete and size of the aggregate,
(6) the type of subgrade or base, and
(7) the temperature and moisture conditions at the time of loading.

To compare the effectiveness of different types of dowel-bar joints, each type would have to be tested on a variety of subgrades and bases and sufficient repetitions made to allow for unavoidable variations in these materials. In order to avoid the considerable amount of full-scale work involved, a laboratory test for single dowel-bars has been devised. Such tests can be carried out under conditions which may be standardized, and the results can then be applied to joints in an actual road. Such tests can also assess the effect of various methods of preventing bond, and the extent to which strength of concrete and size of aggregate affect load-transfer.

The problem of determining the deflexions of dowel-bars was first studied by Grinter [18] and Bradbury [19], who provided theoretical solutions for particular cases and assumed conditions. Later a general mathematical solution was devised by Friberg [20] who based his work upon an exact mathematical solution (devised by Timoshenko and Lessels [21]) of the problem for elastic dowel-bars of infinite length embedded in an elastic mass.

The general equation developed by Timoshenko for the deflexion of one half of a dowel-bar of infinite length in an elastic structure is :

$$y = \frac{e^{-\beta x}}{2\beta^3 E_s I}\left\{P \cos \beta x - \beta M_o(\cos \beta x - \sin \beta x)\right\}$$

where P denotes a concentrated applied load (lb.).
x ,, distance from joint face (inches).
y ,, the corresponding deflexion of the dowel-bar at x (inches).
E_s ,, modulus of elasticity of the dowel-bar (lb./sq. in.).
I ,, moment of inertia of the dowel-bar (inches^4).
M_o ,, bending moment in the dowel-bar at the joint face, which is equal to $-\frac{Pa}{2}$

where a denotes the width of the joint.

β represents a term defined as the relative stiffness of the dowel-bar and expressed by the following constant :

$$\beta = \sqrt[4]{\frac{Gb}{4E_s I}}$$

where G denotes modulus of support (lb./inch^3).
b ,, width of bearing area of dowel-bar (that is, diameter).

From the above equation the deflexion of a dowel-bar of infinite length can be plotted (see *Fig. 5*). The variation in pressure will also be represented by the same curve to a different scale. For a dowel-bar of finite length the deflexions and pressures will be increased, and the pressure at the joint face will become greater. It is obviously desirable for economy to use short dowel-bars but the length must not be reduced to such an extent that the high pressures at the joint face will cause spalling. The problem of length of dowel-bar is considered in the paragraphs dealing with laboratory tests.

From the general equation the following specific equations may be obtained :—

(1) Deflexion of the dowel-bar at the joint face ($= y_o$) :

$$y = y_o \text{ when } x = 0$$

$$y_o = \frac{P\left(1 + \frac{\beta a}{2}\right)}{2\beta^3 E_s I} = \frac{P(2 + \beta a)}{4\beta^3 E_s I}$$

(2) Bending moment at any point in a dowel-bar ($= M$) :

$$M = -E_s I \frac{d^2y}{dx^2}$$

$$= -\frac{e^{-\beta x}}{\beta}\left\{P \sin \beta x - \beta M_o (\sin \beta x + \cos \beta x)\right\}$$

(3) Shearing force at any point in a dowel-bar ($= V$) :

$$V = \frac{dM}{dx}$$

$$= -e^{-\beta x}\{(2\beta M_o - P) \sin \beta x + P \cos \beta x\}$$

The overall deflexion (Δ) across the joint will be made up of :

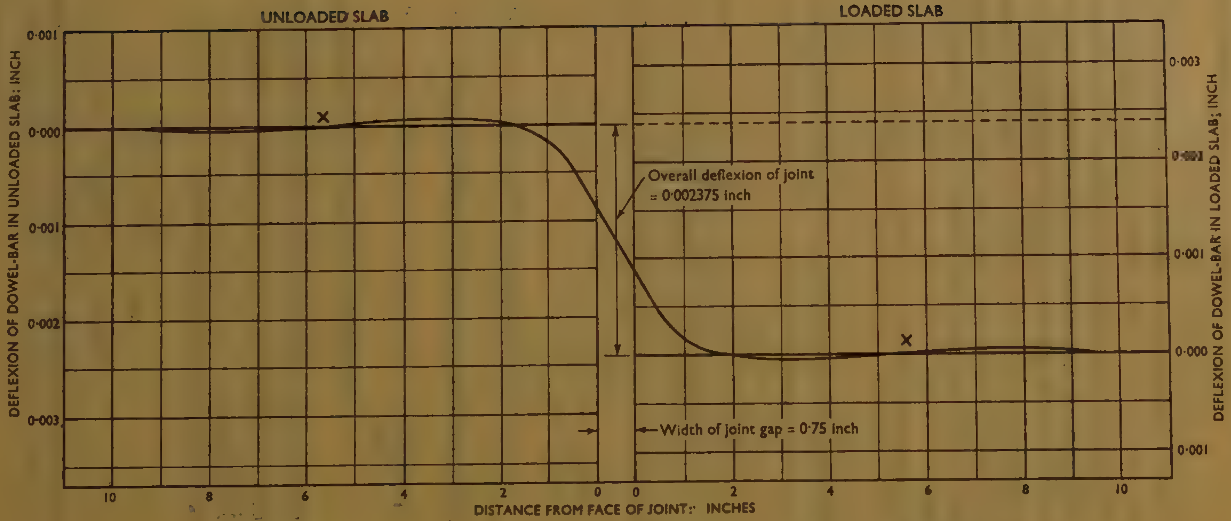
2 × (deflection of dowel-bar in concrete) plus (deflexion of dowel-bar across joint gap).

Now : 2 × (deflexion of dowel-bar in concrete)

$$= \frac{P - \beta M}{\beta^3 E_s I} = \frac{P\left(1 + \beta\frac{a}{2}\right)}{\beta^3 E_s I}$$

and : (deflexion of dowel-bar across joint gap) = (deflexion caused by slope of dowel-bar) plus (deflexioncaused by flexure of dowel-bar across gap)

Fig. 5



PROFILE OF A DOWEL-BAR JOINT LOADED ON ONE SIDE WITH A 1,000-LB. LOAD
(Diameter of Dowel-bar: 1 inch)

$$= a\frac{dy_0}{dx} + 2\frac{P}{3E_sI}\left(\frac{a}{2}\right)^3$$

$$= a\left\{-\frac{P}{2\beta^2E_sI}(1+\beta a)\right\} + 2\frac{P}{3E_sI}\left(\frac{a}{2}\right)^3$$

Therefore : $$\varDelta = \frac{P\left(1+\beta\frac{a}{2}\right)}{\beta^3E_sI} - \frac{Pa(1+\beta a)}{2\beta^2E_sI} + \frac{2P\left(\frac{a}{2}\right)^3}{3E_sI}$$

$$= \frac{P}{2E_sI}\left\{\frac{1+(1+\beta a)^2}{\beta^3} + \frac{a^3}{6}\right\}$$

This equation gives the theoretical relationship between applied load and deflexion of the dowel-bar across the joint. This relationship is the real criterion of the effectiveness of a dowel-bar and in this Paper is referred to as the rigidity of the dowel-bar ; this is defined as the load on a dowel-bar required to produce unit deflexion across the joint or, in terms of the above symbols :—

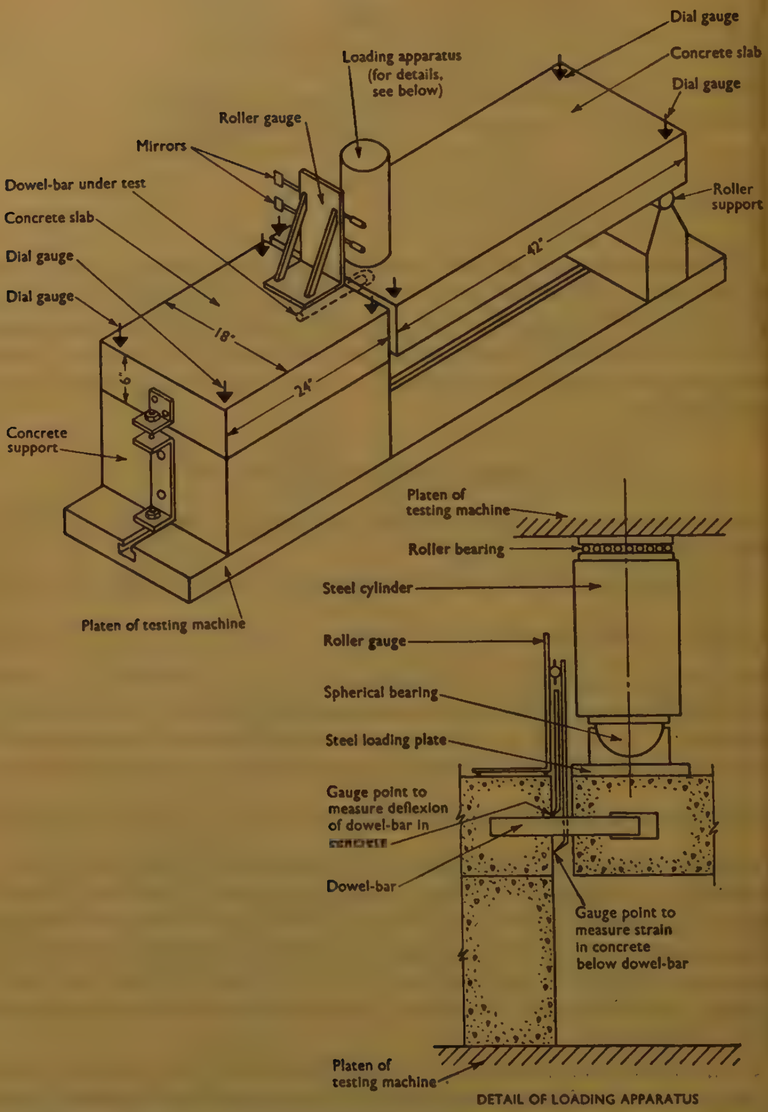
$$\text{rigidity of dowel-bar } (\lambda) = \frac{P}{\varDelta} \text{ (lb. per inch)}$$

It will be seen that λ is thus dependent upon the terms E_s, I, a, and β, which in turn depend on G and b.

All these terms can be readily determined except for the value of G $\left(\text{occurring in the expression } \beta = \sqrt[4]{\frac{Gb}{4E_sI}}\right)$. G has been defined by Friberg as the modulus of support (lb. per inch^3), and values of 300,000 lb. per inch^3 and 1,000,000 lb. per inch^3 have been suggested by Grinter [18] and Friberg [20] respectively. Friberg [22] has also suggested that G is related to the modulus of elasticity of the concrete and to the amount of cover over the dowel-bars, and that it can be determined from the equation $G = \frac{E_c}{d}$, where E denotes the modulus of elasticity of the concrete in compression (lb. per square inch), and d denotes the amount of cover of concrete over the dowel-bar (inches). A further suggestion by Friberg is that $G = 300$ times the cylinder strength of the concrete. From the tests made at the Road Research Laboratory, the average value of G has been found to be 2,500,000 lb. per inch^3, and the theoretical deflexions given in this Paper have been calculated assuming G to have this value.

Of the other terms needed for the calculation of λ, E_s will depend upon the type of steel used, a is the width of the joint, and I and b depend upon the shape and size of dowel-bars. If a dowel-bar of given rigidity is required, the only factors which readily lend themselves to adjustment during design are those affecting I and b, that is, the shape and size of the cross-section of the dowel-bar.

Figs 6


Dial gauge
Concrete slab
Dial gauge
Loading apparatus (for details, see below)
Roller gauge
Mirrors
Dowel-bar under test
Concrete slab
Dial gauge
Dial gauge
Roller support
42"
18"
24"
6"
Concrete support
Platen of testing machine
Platen of testing machine
Roller bearing
Steel cylinder
Roller gauge
Spherical bearing
Steel loading plate
Gauge point to measure deflexion of dowel-bar in concrete
Dowel-bar
Gauge point to measure strain in concrete below dowel-bar
Platen of testing machine
DETAIL OF LOADING APPARATUS

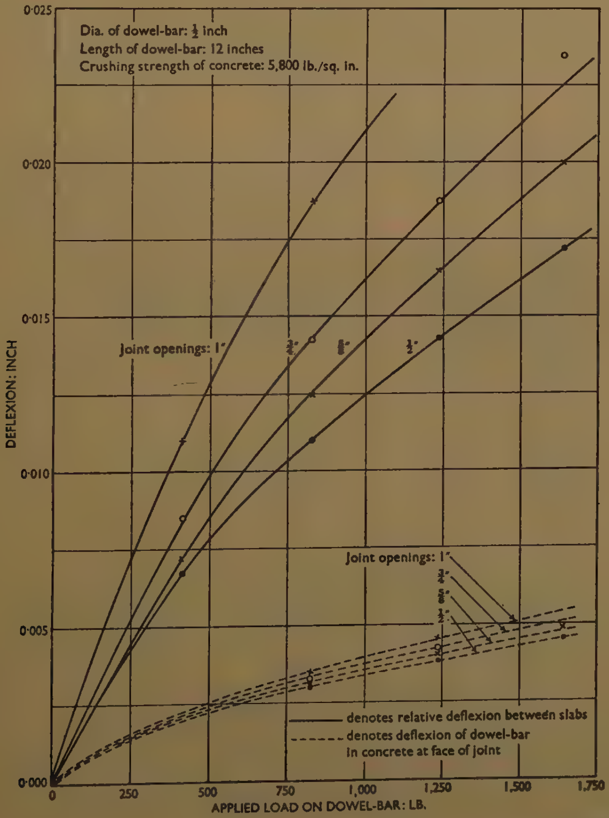
DIAGRAMMATIC VIEW OF ARRANGEMENT FOR CARRYING OUT LABORATORY TESTS ON DOWEL-BARS

Laboratory tests have been made on single dowel-bars in the following manner. Test specimens were made with single dowel-bars set across the joint between two concrete blocks. One of these blocks was supported over the whole of its area on a rigid base and the other pivoted about 3 feet from the joint. The unsupported side of the joint was then loaded in increments and the relative deflexion between the two blocks and the deflexions of the dowel-bar in the concrete were measured. A diagrammatic sketch of the apparatus is shown in *Figs 6*.

With this method of testing, that part of a wheel load which would normally be counterbalanced by the reaction of the subgrade under the loaded slab is eliminated, and the load transferred by the dowel-bar is approximately equal to the load applied to the specimen.

By means of the laboratory test, the effect of (1) diameter of dowel-bar,

Fig. 7

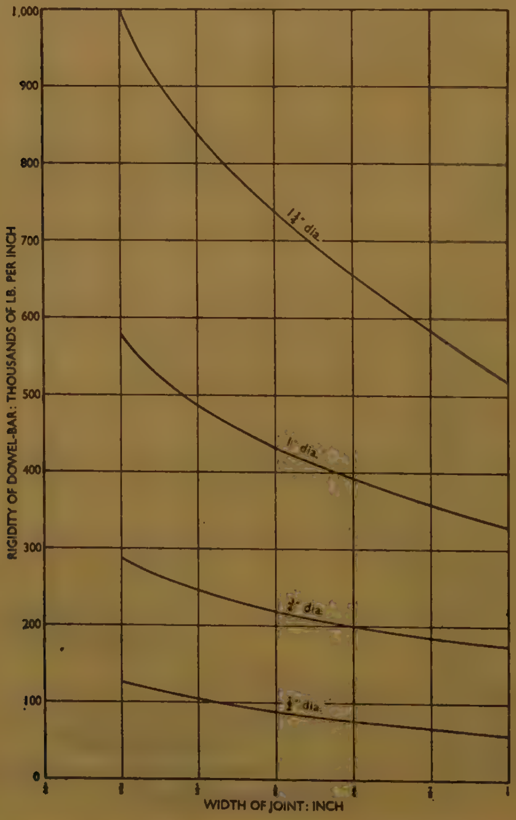


Relation between Relative Deflexion of Slabs and Deflexion of Dowel-Bar in Concrete and Load for Various Joint Openings

(2) length of dowel-bar, and (3) width of joint was investigated. There is a considerable variation in the results which is probably due to loca variations in the properties of the concrete immediately around the dowel-bar at the joint face; these local variations will probably be greater than the variations registered when measuring the crushing strength o test cubes. The particular arrangement of the aggregate particles around the dowel-bar may have an important effect on the results. Similar o larger variations will certainly occur between different dowel-bars in a joint in a road but, since an individual load is spread over several dowel-bars, such variations will tend to be evened out and the relative deflexion between the slabs will approximate to that to be expected from the average value of the rigidity of the dowel-bars.

Typical results from tests on a dowel-bar are shown in *Fig. 7*. To

Fig. 8



THE RIGIDITY OF DOWEL-BARS FOR VARIOUS WIDTHS OF JOINT (VALUES DETERMINED EXPERIMENTALLY) enable the results of similar tests on various dowel-bars to be readily calculated, the rigidity of the dowel-bar (that is, the ratio between load and relative deflexion) is calculated for the first 1,000 lb. of load.

The relationship between the rigidity (λ) of the dowel-bar, diameter of dowel-bar, and width of joint is shown in *Fig. 8*, which shows the average results obtained from laboratory tests on dowel-bars 12 to 18 inches long, coated with a thin layer of bitumen. Dowel-bars of 9-, 12-, 18-, and 24-inch lengths were tested; the length, within this range, had no effect on the rigidity of the dowel-bar. Friberg suggested that the length of the dowel-bar could be limited to the distance between the second points of reversal of pressure (points X in *Fig. 5*) without causing undue increase of pressure at the joint face. The distances between these points on each side of the joint have been calculated and are given in Table 1. To allow for errors in alignment and for the possibility of variation in the compaction and quality of the concrete under site conditions, it is suggested that these lengths should be increased by 75 per cent and that a further 3 inches should be added to allow for the dowel-bar cap and for poorer contact over the sliding half of the bar.

TABLE 1.—SUGGESTED MINIMUM LENGTHS OF DOWEL-BAR (CALCULATED FOR JOINTS HAVING A WIDTH OF ¾ INCH AND FOR A MODULUS OF SUPPORT OF 2,500,000 LB./SQ. IN./IN.)

Diameter : inch	Total length between second points of contraflexure (calculated from Friberg's analysis) : inches	Suggested total length (length between second points of contraflexure + 75 per cent + 3 inches allowance for cap and sliding half) : inches
½	7¼	16
¾	9¾	20
1	12	24
1¼	14	28

No definite relationship between the results and variations in crushing strengths or thickness of film of bitumen could be obtained but to investigate these factors a further series of laboratory tests is being made with simpler specimens in which only one half of the joint is tested.

Although the laboratory tests have been carried out on solid round bars, tubular, square, or rectangular sections could be used. The rigidity of such bars has been estimated from their section moduli and is given in Table 2 for two different areas of cross-section. It will be seen that tubular sections with thin walls, and rectangular sections with a large height/width ratio, have the greatest rigidity for a given amount of steel; of these two sections the tubular one is preferable because of its greater bearing area and hence higher failing load; solid round bars are, however cheaper and easier to handle.

TABLE 2.—ESTIMATED DOWEL-BAR RIGIDITY FOR SECTIONS OF DIFFERENT SHAPE BUT WITH THE SAME CROSS-SECTIONAL AREA (ESTIMATED FOR A JOINT ¾ INCH WIDE)

Type	Cross-sectional area of bar: square inches			
	0·196		0·442	
	Size: in.	Rigidity: lb./in.	Size: in.	Rigidity: lb./in.
Solid round	0·50 dia.	75×10^3	0·75 dia.	200×10^3
Tubular (⅛ inch thick) . .	0·62 o/dia.	110×10^3	1·25 o/dia.	370×10^3
Tubular (¼ inch thick) . .	—	—	0·81 o/dia.	230×10^3
Square	0·44	90×10^3	0·67	225×10^3
Rectangular (Height = 1½ × width)	0·54 × 0·36	100×10^3	0·82 × 0·54	265×10^3
Rectangular (Height = 2 × width)	0·63 × 0·31	115×10^3	0·74 × 0·47	300×10^3

ASSESSMENT OF EFFECTIVENESS OF LOAD-TRANSFERENCE

In order to compare the effectiveness of load-transfer of various sizes and spacings of dowel-bar and other types of joints, it is necessary to have some measurable basis for comparison. Two convenient ways of doing this are to measure the stresses or the deflexions in the loaded and unloaded slabs. Complete efficiency may then be defined as the state when the stresses and deflexions in the loaded and unloaded slabs are equal, and zero efficiency as that when the stresses and deflexions in the unloaded slab are zero, thus indicating that no load is being transferred.

At the edge or corner of a slab, the stresses and deflexions are approximately proportional to the load. The efficiency of a joint is therefore approximately similar whether it is determined on a basis of stress or deflexion. Since deflexions can be more readily measured than stresses, especially where measurements are made on existing pavements, the load-transfer efficiencies given in this Paper are in terms of deflexions, and are calculated from the following formula:—

$$\eta = \frac{\delta_u}{\frac{1}{2}\delta_e} \times 100 \text{ per cent}$$

where η denotes load-transfer efficiency of joint,

δ_u ,, deflexion of unloaded slab at joint due to a load (W) on the adjacent slab at the joint, and

δ_e ,, deflexion of free edge of a similar slab due to a load (W).

Assuming that the deflexion is proportional to the load, the efficiency may be expressed in terms of δ_u and δ_e as follows:

$$\eta = \frac{\delta_u}{\frac{1}{2}\delta_e} \times 100 \text{ per cent} = \frac{2\delta_u}{\delta_u + \delta_l} \times 100 \text{ per cent}$$

where δ_l denotes deflexion of loaded slab at joint.

Another method of measuring the effectiveness of load-transfer is to compare either the stresses or deflexions at the joint with those at a point away from the edges of the slab. Full efficiency then becomes the condition in which the stress or deflexion at the loaded side of the joint is no greater than at the interior of the slab. This method, using stress as a basis of comparison, is used by Teller and Sutherland [23], but is not adopted in this Paper because it is thought preferable to have a measure of joint efficiency in which full efficiency corresponds to the maximum attainable efficiency—in other words, that given by a perfectly rigid load-transfer device. The formula given above is suitable for determining the load-transfer efficiency of a joint when the load is on one slab only. As the most severe condition will occur when the load is at the edge of the slab, but is still wholly supported by it (that is, when the contact area is tangential to the joint), this condition only is considered in this Paper, but the efficiency of a joint with the load partly on each slab may be determined by a method given by Westergaard.[6]

The maximum stress resulting from loading at the free edge of the slab may be anything up to about three times the corresponding stress at the interior of the slab. This increase in stress at the edges will be partly counterbalanced by reduced temperature-warping-stresses but even so there will still be a considerable increase in the stress at the edges which makes any means of reducing the stress very desirable.

The reduction in stress obtained by a load-transfer device may be determined as follows. If the efficiency is η, then the amount of load transferred $= \frac{W}{2}\eta$ where W denotes the applied load. The stress at the joint in the loaded slab will therefore be:—

$$\sigma_j = \sigma_e \left(1 - \frac{\eta}{2}\right)$$

where σ_j denotes stress in the loaded slab at the joint σ_e denotes stress in a loaded free edge. Taking as an example a wheel load of 75,000 pounds on a slab 12 inches thick, then:

stress at interior = 230 lb./sq. in.
stress at joint with "perfect" load-transfer device ($\eta = 100$ per cent) = 315 ,,
stress at joint where $\eta = 90$,, ,, = 350 ,,
,, ,, ,, ,, $\eta = 80$,, ,, = 380 ,,
,, ,, ,, ,, $\eta = 70$,, ,, = 410 ,,

stress at joint where $\eta =$ 50 per cent = 470 lb./sq. in.
" " " " $\eta =$ 30 " " = 540 "
" " " with no load-transfer device (or
at a free edge) = 630 "

The maximum temperature warping stress in a slab of this thickness would be about 400 lb. per square inch. Assuming that this is reduced by about one-third at the slab edges, the total stresses at a joint of 80-per-cent efficiency will be about 650 lb. per square inch, compared with 630 lb. per square inch at the interior. With stresses of this order there is very little margin between stress and ultimate strength and it is therefore very desirable to reduce loading stress as much as possible by high efficiency of load transference. Dowel-bar joints should normally be designed to give an efficiency of about 80 per cent; such efficiencies can generally be obtained from dowel-bars of moderate size except on very rigid bases where dowel-bars of large diameter may be required. Efficiencies of below 60 per cent hardly justify the extra cost of the dowel-bars.

The amount of relative vertical movement between the slabs must also be considered, since excessive relative movements will subject the filling and sealing materials to considerable strain in shear, which may increase the rate of their deterioration; it will also produce impact between the wheel and slab, which may cause spalling of the joint edges and impair the riding quality of the pavement. Although there are no data which would suggest a limiting value for the relative movement, a dowel-bar joint with a load-transfer efficiency of 80 per cent would theoretically reduce the relative movement to one-third of that for an undowelled joint; experiments described later (see *Figs 18*, p. 644) have shown that the reduction may be even greater than this amount.

In considering load-transfer efficiencies no account has been taken of the joint-filling material, which may behave as a load-transfer device of considerable efficiency if the joints are tightly butted together in hot weather. Efficiencies calculated from slab deflexions occurring during the summer months may therefore be greater than expected but should not be taken into account for design purposes, because during the winter, or even during a summer night, the load transferred by the filling material will probably be negligible.

The Effect of Type and Spacing of Dowel-Bars on the Performance of a Joint

The results of theoretical examination and laboratory tests on single dowel-bars now have to be applied to the case of actual joints. If the load-deflexion relationship is known, the load-transfer efficiency of a dowel-bar joint may be calculated for any condition if certain assumptions are made. Such a method has been produced by Westergaard[24] in respect of a joint with four dowels only, with the load applied centrally; but the reactions of any dowel-bars outside the four considered are quite considerable and the following method is suggested as being preferable. It is based upon the same assumptions as the formula expressing load-transfer efficiency mentioned previously, but with the additional premise that the force transmitted by any dowel-bar is proportional to the deflexion of the unloaded slab at that point.

The deflexions at various points along the free edge of a loaded slab have to be known, and the theoretical deflexion curve for edge-loading conditions, devised by Westergaard [25], is used in the following calculations; however, any other theoretically or experimentally obtained curve could be used. From the above assumptions, it may be shown that:

$$\eta = \frac{1}{1 + \frac{m}{2\lambda} \cdot \frac{Kl^2}{A}}$$

where η denotes efficiency of load-transference of joint,

λ ,, rigidity of dowel-bar (lb./in.),

K ,, modulus of subgrade reaction (lb./sq. in./in.),

l ,, radius of relative stiffness (inches),

$m = \frac{as}{A}$.

A denotes area under deflexion curve (square inches),

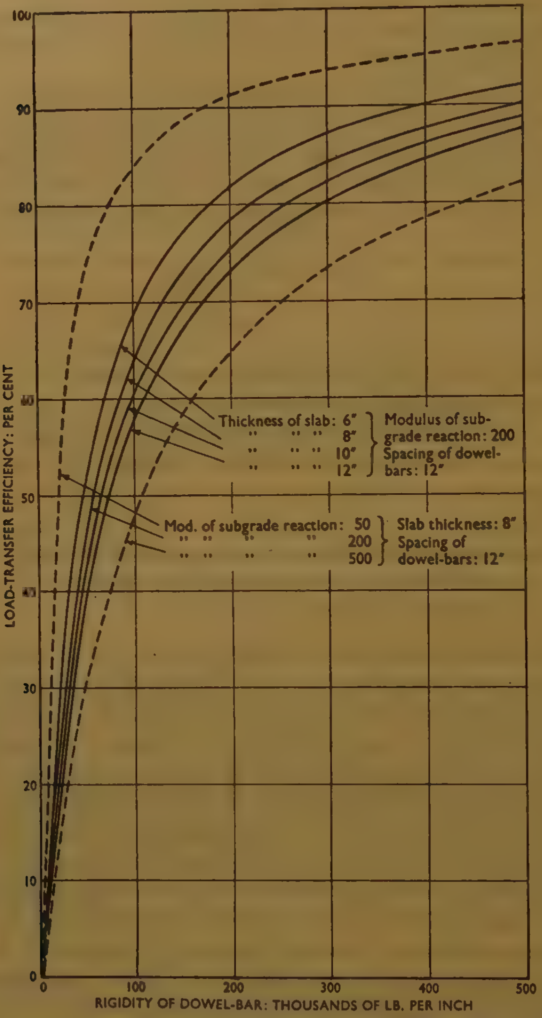
a ,, deflexion of a single dowel-bar (inches),

s ,, spacing of dowel-bars (inches).

From the above formula the relationship between load-transfer efficiency and rigidity of the dowel-bar has been calculated for various thicknesses of slab and for subgrades of various qualities, and is shown graphically in *Fig. 9*. It will be seen that the efficiency increases very rapidly with an increase in the rigidity of the dowel-bar up to a value for λ of about 200,000, but beyond this value the increase in efficiency is fairly small.

It will also be seen that for a given spacing of bars, the thicker slabs or more rigid subgrades require a greater rigidity of dowel-bar. This may at first seem illogical, but it is because of the smaller deflexions (under a given load) of the thicker slabs or more rigid subgrades, and the consequent need for a more rigid bar to transmit a given proportion of the load, thus maintaining a given load-transfer efficiency. With very thick and rigid constructions a point is probably reached at which there is little advantage in providing dowel-bars for load-transfer purposes; for this reason it was suggested in the Introduction that dowel-bars would be of greater value in pavements to be constructed on poor soils, or in pavements carrying loads which do not warrant a very rigid construction, than in the very thick and rigid pavements of the main airports. In the latter type of airfield, however, dowel-bars still serve the useful purpose of providing smoother riding over joints (thus reducing impact), and of reducing rocking of the slabs.

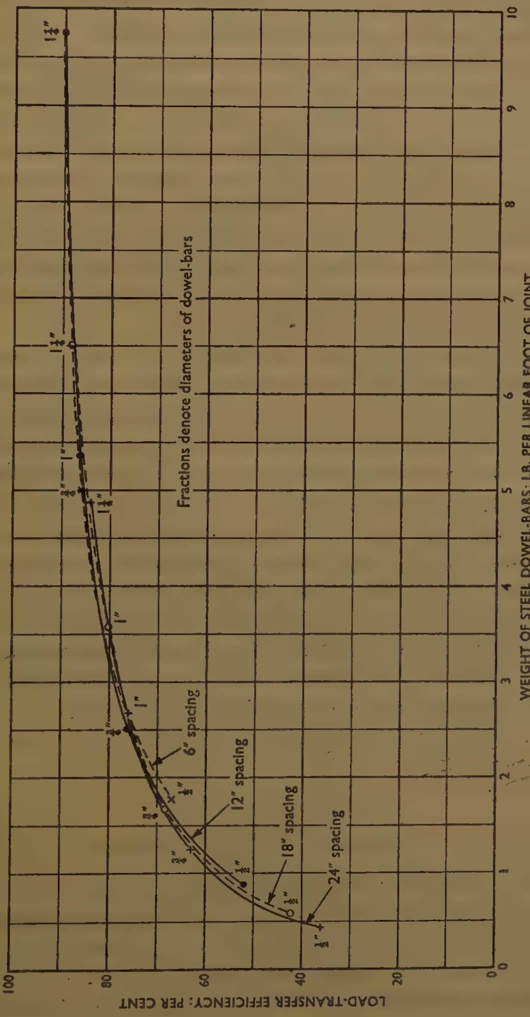
Fig. 9



Relationship between Load-Transfer Efficiency and the Rigidity of the Dowel-Bar

The size and spacing of dowel-bars for various conditions may now be considered. Ideally, a load-transfer device would provide continuous and uniform support along a joint. The rigidity of a concrete slab makes it possible, however, for effective load-transference to be obtained by a series of bars spaced at intervals along the joint, and by this means a given degree of load-transference may be obtained with greater economy in the use of steel than would be the case if the device had to be continuous. For given conditions there is usually a choice between large-diameter dowel-bars at wide spacings or smaller bars at closer spacings and it might at first appear that the greatest economy in the use of steel would be obtained by using the large bars at wide spacings. In a given joint the deflexion

Fig. 10



RELATIONSHIP BETWEEN LOAD-TRANSFER EFFICIENCY AND WEIGHT OF STEEL DOWEL-BARS (LENGTH OF DOWEL-BARS : AS IN TABLE 1 ; WIDTH OF JOINT : 1 INCH) of the bar across the joint gap will depend upon its moment of inertia and the deflexion of the bar in the concrete will depend largely upon the bearing area of the bar in the concrete. The overall deflexion is thus partly proportional to the fourth power of the diameter and partly to the diameter. The relationship between load-transfer efficiency and the weight of steel per linear foot of joint has been calculated from the results of laboratory tests and is shown graphically in *Fig. 10*, where allowance has been made for the greater length required in the larger-diameter dowel-bars in order to prevent excessive pressure on the concrete at the joint face. It will be seen that for a given amount of steel there is little difference in the load-transfer efficiency of dowels spaced at 6-, 12-, 18-, and 24-inch centres. Two considerations limit the range of spacings: (1) for simplicity of construction the number of dowel-bars must be kept to the minimum (it is therefore preferable not to use spacings closer than 12 inches), and (2) the flexure of the concrete slabs between the dowels must be kept to the minimum (for this reason the spacings should not exceed 24 inches).

The size and spacing of dowel-bars needed to obtain a given amount of load-transference may now be determined for various subgrades, thicknesses of slab, and widths of joint. The values given in Table 3 have been calculated for a load-transfer efficiency of 80 per cent as suggested earlier. They are based on the results obtained from the laboratory tests and are applied to the case of an actual joint in a concrete pavement by means of the method already described. The various sizes are given for the nominal width of the joint, that is, the width of the joint as constructed, but to allow for opening and closing of the joint the sizes have been calculated for a joint $\frac{1}{4}$ inch wider than the nominal width. The 12-inch spacing is usually preferred. At the corners of slabs the reactions in the dowel-bars will increase, and it is suggested that from a point about 4 feet from the corner the spacing between neighbouring dowel-bars be decreased progressively down to half the normal spacing between the two dowel-bars nearest the corner.

THE EFFECT OF WARPING ON THE BEHAVIOUR OF A JOINT

Warping of a concrete slab is produced by temperature or moisture gradients through its depth, which cause differential expansion and contraction. When the top of the slab is hotter or in a wetter condition than the bottom of the slab, the edges and corners will warp downwards thus tending to lift the whole slab as shown by curve (1) in *Fig. 11* where an actual profile of a warped slab is reproduced. The weight of the slab limits the amount of warping but not to the extent which might be expected, and it is probable that deformation of subgrade accommodates some of the change in shape of the slab.

When the top of the slab is cooler or drier than the bottom, the edges and corners will warp upwards, thus reducing the contact pressure between

TABLE 3.—DIAMETERS AND SPACINGS OF DOWEL-BARS REQUIRED TO GIVE A LOAD-TRANSFER EFFICIENCY OF NOT LESS THAN 80 PER CENT (A STRESS REDUCTION OF 40 PER CENT) FOR VARIOUS CONDITIONS

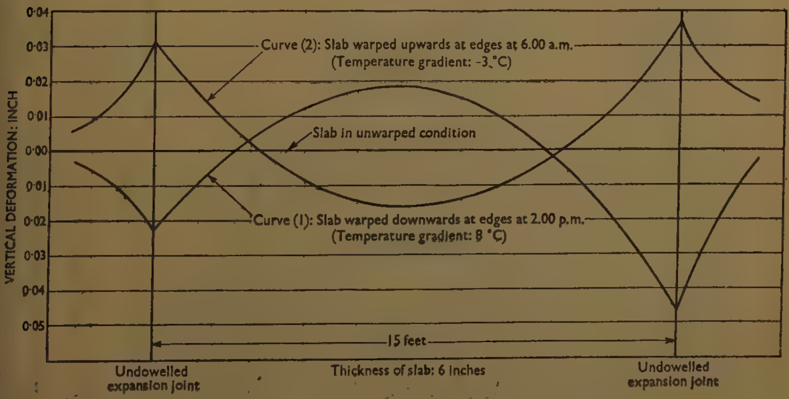
(The lengths of the dowel-bars should be as in Table 1)

Nominal width of joint : inch	Spacing of dowel-bars : inches	Modulus of subgrade reaction : lb./sq. in./in.											
		50				200				500			
		Thickness of concrete slab : inches											
		6	8	10	12	6	8	10	12	6	8	10	12
0	12	½	½	½	½	¾	¾	¾	¾	¾	1	1	1
	18	½	½	½	¾	¾	¾	1	1	1	1	1	1¼
	24	½	½	¾	¾	¾	1	1	1	1	1	1¼	1¼
½	12	½	¾	¾	¾	¾	1	1	1	1	1¼	1¼	1¼
	18	¾	¾	¾	¾	1	1	1¼	1¼	1¼	1¼	(1½)	(1½)
	24	¾	¾	¾	¾	1	1¼	1¼	1¼	1¼	(1½)	(1½)	(1½)
¾	12	¾	¾	¾	¾	1	1	1	1	1¼	1¼	1¼	(1½)
	18	¾	¾	¾	¾	1	1¼	1¼	1¼	1¼	(1½)	(1½)	(1¾)
	24	¾	¾	¾	1	1¼	1¼	1¼	1½	(1½)	(1½)	(1¾)	(1¾)
1	12	¾	¾	¾	¾	1	1	1¼	1¼	1¼	(1½)	(1½)	(1¾)
	18	¾	¾	¾	1	1¼	1¼	1¼	1½	(1½)	(1¾)	(1¾)	(2)
	24	¾	¾	1	1	1¼	1¼	1½	1½	(1¾)	(1¾)	(2)	(2)

Figures in parentheses are extrapolated results outside the range of the laboratory tests.

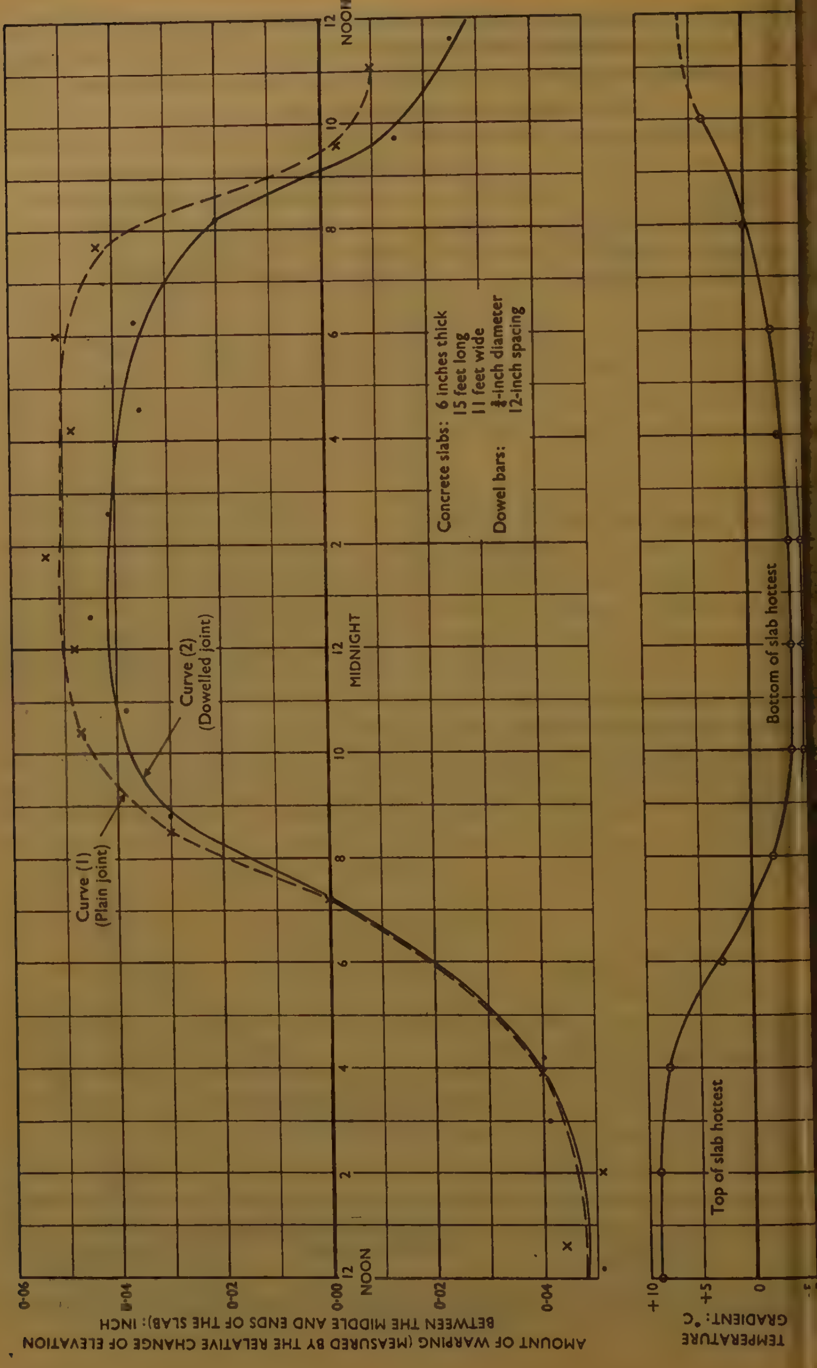
NOTE.—The size of the dowel-bar required for given conditions must be adequate to meet the requirements of load-transfer efficiency given above and the requirements of strength given in Table 5. Where the sizes differ, the larger value of the two should be used.

Fig. 11

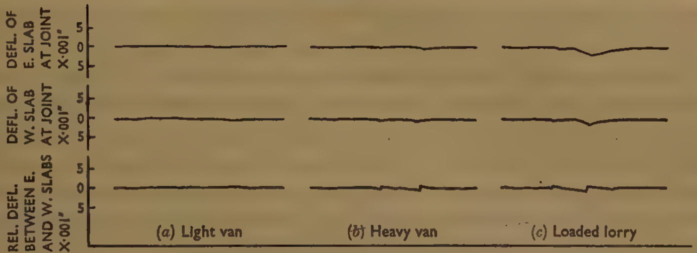


TYPICAL PROFILES OF A WARPED CONCRETE SLAB AS MEASURED ON 2ND-3RD AUGUST, 1951

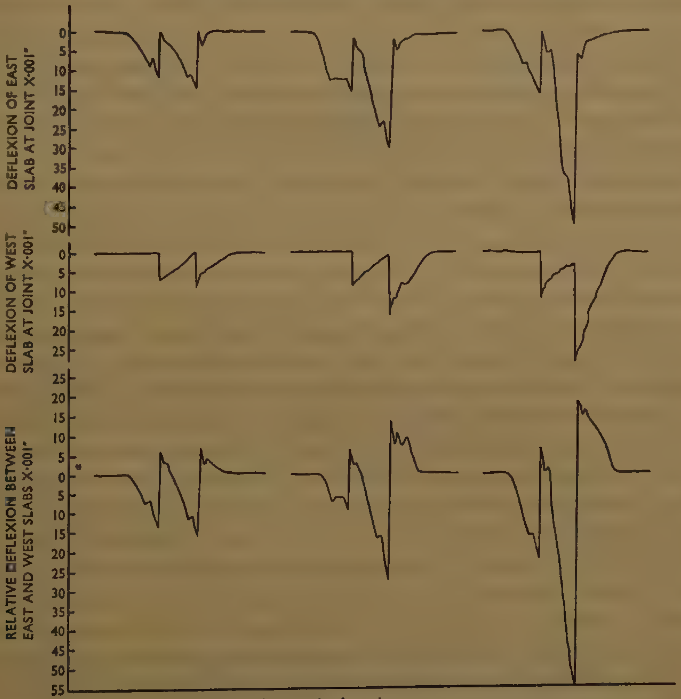
Figs 12


12 NOON
10
8
6
4
2
12 MIDNIGHT
10
8
6
4
2
12 NOON
Concrete slabs: 6 inches thick
15 feet long
11 feet wide
Dowel bars: ¾-inch diameter
12-inch spacing
Curve (1) (Plain joint)
Curve (2) (Dowelled joint)
0·06
0·04
0·02
0·00
0·02
0·04
AMOUNT OF WARPING (MEASURED BY THE RELATIVE CHANGE OF ELEVATION BETWEEN THE MIDDLE AND ENDS OF THE SLAB): INCH
Bottom of slab hottest
Top of slab hottest
+10
+5
0
TEMPERATURE GRADIENT: °C

Figs 14



(1) During day-time (at 3 p.m.)
Temperature gradient of +13 °C from top to bottom of slab



(2) During night-time (at 6 a.m.)
Temperature gradient of -5 °C from top to bottom of slab

	Light van	Heavy van	Loaded lorry
Weight of front axle	1,400 lb.	2,700 lb.	3,100 lb.
Weight of rear axle	1,500 lb.	4,200 lb.	13,500 lb.
Slab: 8" unreinforced concrete on compacted gravel base			

DEFLEXIONS DUE TO LOADING AT AN UNDOWELLED JOINT DURING TYPICAL DAY AND NIGHT CONDITIONS FOR EARLY SUMMER

(Measured by means of electronic apparatus developed by J. J. Trott.[26])

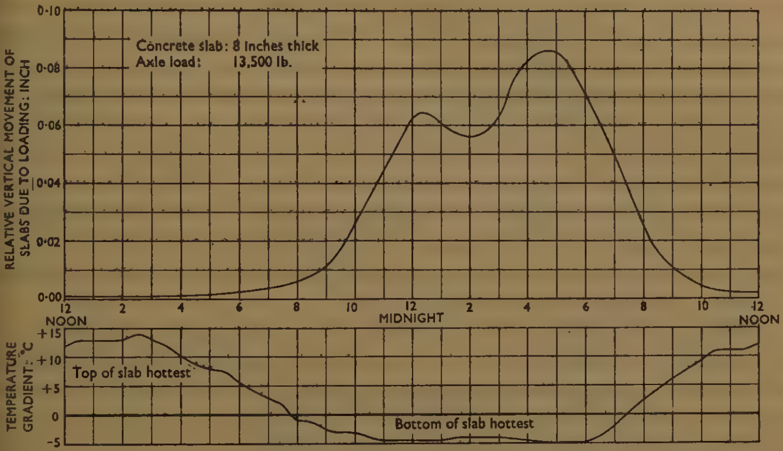
slab and subgrade or base at these positions or even lifting part of the slab clear of its support (an actual profile is shown as curve (2) in *Fig. 11*). Under these conditions the only restraint on the warping of short slabs will be the weight of the edges and corners of the slab. Upward warping usually occurs at night and is usually greatest in early summer when a hot sunny day is followed by a clear cool night, and rapid cooling of the surface by radiation occurs. For such conditions, a typical graph showing the variation in the amount of warping over a 24-hour period is given by curve (1) in *Figs 12,* where the amount of warping is indicated by the relative change of elevation between the middle and edge of the slab.

When the slab is warping upwards at the edges and corners the vertical deflexion of a joint under loading will be considerably greater than under normal conditions. This was shown by a series of 24-hour tests in which the amount of warping, the widths of the joint, the temperature gradient, and the absolute and relative vertical deflexions of the slabs under loads were measured. The absolute deflexions of the slab were measured with reference to large concrete blocks which were constructed at the side of the joints to be tested but clear of the slab. Both the absolute and relative movements were measured by electronic apparatus devised by Trott[26] and illustrated in *Fig. 13*. Typical records obtained during the day and night for the passage of a vehicle over an undowelled joint in an 8-inch slab are reproduced in *Figs 14*. The deflexion of the slab on the side of the joint from which the vehicle is approaching is shown by the top graph, the deflexion of the far side of the joint by the middle graph, and the relative deflexions between the two slabs by the bottom graph. It will be seen that the deflexions occurring during the night are many times larger than those occurring (under a similar vehicle) during the day, and such large movements, as already mentioned, will cause greater stresses, detract from the riding quality, cause excessive wear on the filling and sealing materials, and may result in spalling of the joint owing to impact. The variation in the deflexions caused by loading over a 24-hour period are indicated in *Figs 15* where the relative deflexions of an undowelled joint under the rear axle of a vehicle are plotted. It will be seen that the increase in deflexion corresponds with the reversal of temperature gradients, and during this period the stresses due to loading of the slab will be considerably increased.

The importance of these increased stresses will depend partly upon the length of time during which they occur, and some information on this point has been obtained from an examination of the records of temperatures in a concrete slab, 9 inches thick, for a period of 12 months. The total period of time during which various temperature gradients occur has been found and the results are summarized in *Fig. 16*.

It will now be considered whether dowel-bars can assist in reducing the stresses at the edge and corners of a slab when it is warped upwards and whether they will reduce the relative movement between the two slabs. The moment of resistance of a dowel-bar in bending is small compared with

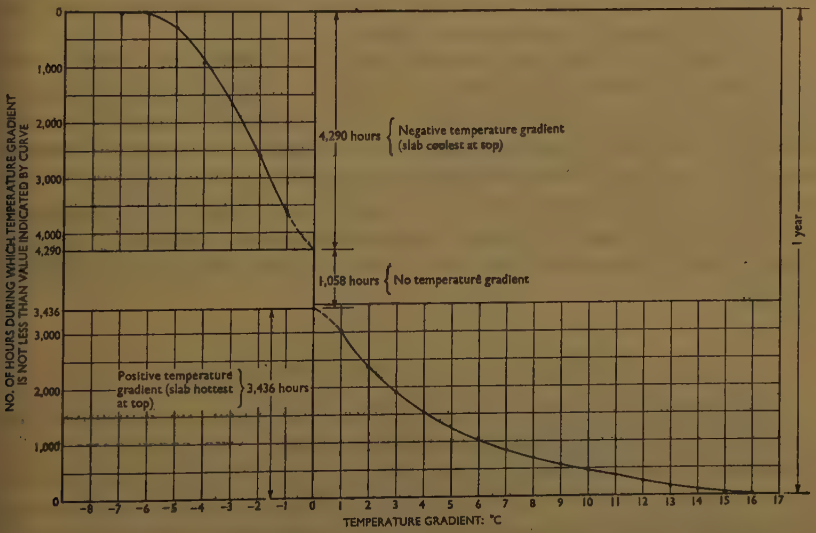
Figs 15



VARIATION IN RELATIVE MOVEMENT DUE TO LOADING WITH TEMPERATURE GRADIENT FOR A 24-HOUR PERIOD

(Measured by means of electronic apparatus developed by J. J. Trott.[26])

Fig. 16



LENGTH OF TIME IN ONE YEAR DURING WHICH TEMPERATURE GRADIENTS OF VARIOUS MAGNITUDES OCCURRED

(Taken from records for a concrete slab, 9 inches thick, between 1st Jan. and 31st Dec. 1940.)

the moment of resistance of a concrete slab, and thus its effect in reducing the amount of warping will be small, but it does have some effect, as is shown by *Figs 12.* This shows the variation in the amount of warping as indicated by the difference in movement between the centre and edge of the slab, occurring during a period of 24 hours in two slabs, similar in all respects except that one has plain joints and one dowelled joints. Loading tests were made during the same 24-hour period and typical records for day and night are reproduced in *Figs 17.* In *Figs 18* the variation in the relative deflexions of the plain and dowelled joints resulting from loading are given for the 24-hour period, and it will be seen that the dowel-bars have reduced the relative movement of the slab during upward warping by a very considerable amount.

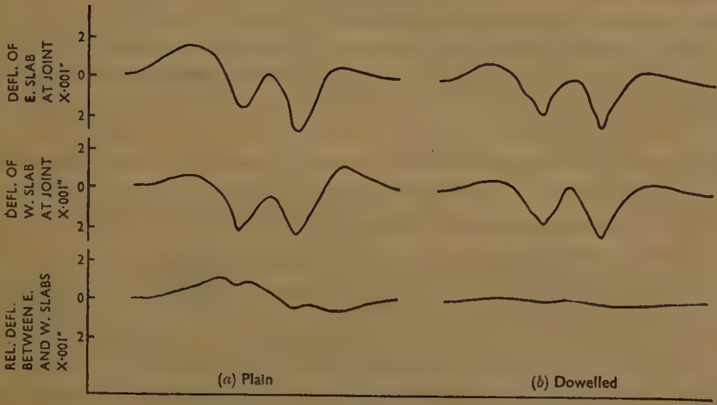
There are two other conditions which may result in the slabs behaving in a manner somewhat similar to warped slabs, and in which dowel-bars will assist in reducing the movement at joints. The first occurs when the slabs have set under a temperature gradient, such as may happen on a hot day when the temperature at the top of the setting concrete is lower than at the bottom: when the concrete sets and the temperature gradient disappears, the slab will then have its edges and corners warped upwards. This problem has been investigated by Rhodes [27] but it is not known whether it is a condition which occurs frequently. The second condition may occur if the subgrade has settled unevenly or if mud-pumping has removed subgrade material from the vicinity of the joint: this would lead to rocking of the slab during the passage of a vehicle.

The liability of the slabs to rock during the passage of a wheel load will become greater as the ratio of the load to the weight of the slab increases and is thus more important with very heavy loads. Although rocking is most likely to occur when, for some reason, the edges and corners are in poor contact with the subgrade, some slight rocking may occur when the slab is in good contact with the subgrade, particularly if the subgrade is poor. Dowel-bars will assist in preventing such rocking by adding part of the weight of the adjacent slabs to counterbalance any lifting action.

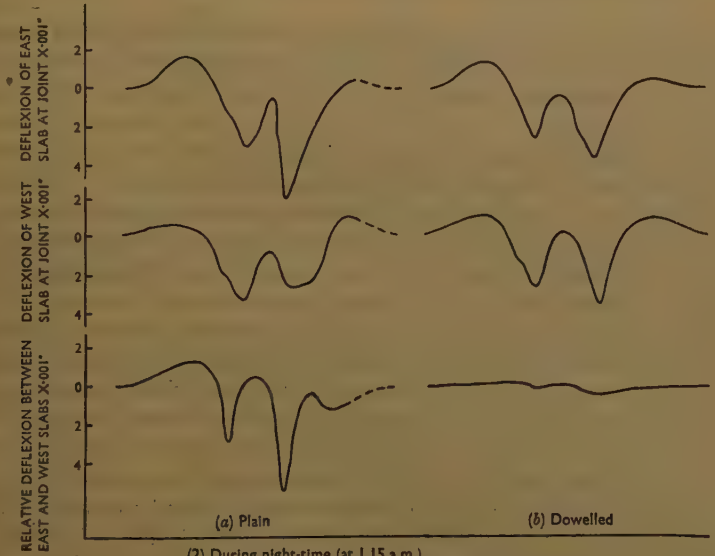
THE STRENGTH OF A DOWEL-BAR JOINT

The ultimate strength of a dowel-bar joint must be considered in addition to its effectiveness in transferring load. Two cases have to be considered, first, the normal case in which a wheel load is applied on one side of a joint and the load transmitted is distributed over a number of dowel-bars, with the bars immediately under or nearest to the point taking the largest part of the force. The second case is that in which settlement has taken place and one slab is partly supported at its edge by the dowel-bars; the total force transmitted by the bars will probably be considerably greater than in the first case but will be spread over a large

Figs 17



(1) During day-time (at 3.45 p.m.)
Temperature gradient of +3½ °C from top to bottom of slab



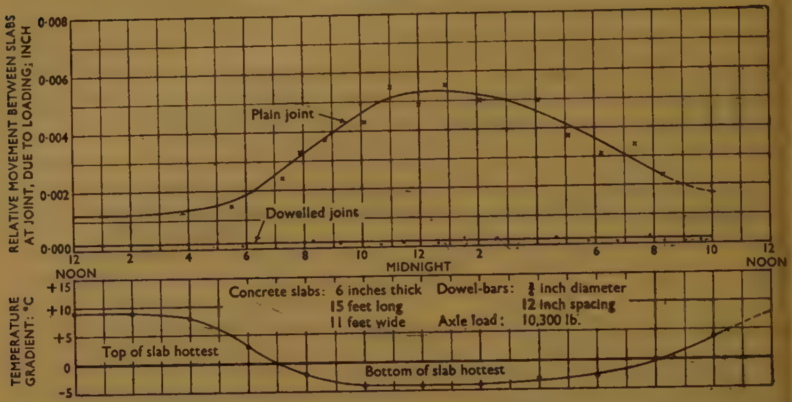
(2) During night-time (at 1.15 a.m.)
Temperature gradient of -4 °C from top to bottom of slab

Slab thickness:......6 inches
Dowel-bars:.........¾″ dia. @ 12″ ctrs
Weight of vehicle: 19,200 lb.

Deflexions due to Loading at Plain and Dowelled Joints during Typical Day and Night Conditions in Summer

(Measured by means of electronic apparatus developed by J. J. Trott.[26])

Figs 18



VARIATION IN RELATIVE MOVEMENT AT PLAIN AND DOWELLED JOINTS DUE TO LOADING UNDER VARIOUS TEMPERATURE GRADIENTS THROUGHOUT A 24-HOUR PERIOD.

(Measured by means of electronic apparatus developed by J. J. Trott.[26])

number of bars. It is impossible to calculate with any degree of accuracy the forces in dowel-bars arising out of such conditions but it is probable that the increased force transmitted will be counterbalanced by the wider spread of the load.

A dowel-bar joint may fail either by bending of the bar or by crushing of the concrete at the point where pressure is highest, that is, close to the face of the joint. The type of failure which occurs in a particular case will be determined by the relations between the rigidity of the dowel-bar and width of joint, and the strength of the concrete. Experiments have shown that bars greater than ½ inch diameter in concrete of moderate to high strength are not likely to fail by bending. Failing loads for dowel-bars of various diameters are given in Table 4, together with suggested working loads. The factor of safety may seem small in view of the likely variations in strength between individual dowel-bars but if one dowel-bar in a loaded joint is weaker than the rest, then the forces between the dowel-bars will be redistributed, and the force on the weaker bar will be reduced.

The force in any individual dowel-bar will depend upon the applied load, the position of the dowel-bar in relation to the load, the load-transfer efficiency of the joint, the spacing of the dowel-bars, the thickness of the slab, and the modulus of subgrade reaction. The force transmitted by the dowel-bar immediately under the load will be greater for wide spacings than for close spacings of the bars, and will be greater where the slab thicknesses and modulus of subgrade reaction are such that the load is spread over a small area, than when it is spread over a large area. Thus the thinner slabs or the higher values of the modulus of subgrade reaction, both of which tend to concentrate the reaction to a load over a small area, will produce a larger force in the dowel-bar immediately under the load than in other cases. The maximum forces in dowel-bars have been calculated for selected values of the variables and the minimum size of dowel-bar adequate to withstand these forces is given in Table 5, using the suggested working loads given in Table 4.

TABLE 4.—FAILING LOADS OF DOWEL-BARS AND SUGGESTED WORKING LOADS

Diameter of dowel-bar: inch	Failing load: tons	Suggested working load: tons
½	1·5	¾
¾	3·1	1½
1	4·5	3
1¼	8·3	6½

TABLE 5.—MINIMUM DIAMETER OF DOWEL-BAR REQUIRED TO WITHSTAND FAILURE UNDER VARIOUS CONDITIONS (FOR A JOINT HAVING A LOAD-TRANSFER EFFICIENCY OF 80 PER CENT)

Wheel load: lb.	Spacing of dowel-bars: inches	Modulus of subgrade reaction: lb./sq. in./in.											
		50				200				500			
		Thickness of concrete slab: inches											
		6	8	10	12	6	8	10	12	6	8	10	12
25,000	12	—	¾	¾	¾	—	¾	¾	¾	1	¾	¾	¾
	18	—	¾	¾	¾	—	1	¾	¾	1	1	1	¾
	24	—	¾	¾	¾	—	1	1	¾	1¼	1	1	1
50,000	12	—	—	¾	¾	—	1	1	1	—	1	1	1
	18	—	—	1	1	—	1¼	1	1	—	1¼	1¼	1
	24	—	—	1	1	—	1¼	1¼	1	—	(1½)	1¼	1¼
75,000	12	—	—	1	1	—	—	1	1	—	—	1¼	1
	18	—	—	1	1	—	—	1¼	1¼	—	—	(1½)	1¼
	24	—	—	1¼	1¼	—	—	(1½)	1¼	—	—	(1½)	(1½)

— indicates thicknesses of slab likely to be insufficient for the load to be carried.

Figures in parentheses are extrapolated results outside the range of the laboratory tests.

NOTE.—The size of dowel-bar required for given conditions must be adequate to meet the requirements of strength given above and the requirements of load-transfer efficiency given in Table 3. Where the sizes differ, the larger value of the two should be used.

The Construction of Dowel-Bar Joints

It is essential that the dowel-bars in a joint shall be able to slide freely, thus permitting the joint to open and close without undue restriction, otherwise the main purpose of the joint will not be achieved. It has been found that a single dowel-bar which, by corrosion, has become bonded into the concrete over the sliding end, may resist sliding with a force of up to 250 pounds per square inch (4½ tons for a dowel 1 inch in diameter by 24 inches long), but a new and smooth bright-mild-steel bar, covered with a uniform layer of bitumen, will offer a resistance of less than 2 pounds per square inch. In order to ensure that such free sliding can take place the surface of the dowel-bar must be smooth, such as is obtained with bright mild steel, and the ends should be clearly sawn so that no burrs are left which would restrict movement. The bond may be broken either by giving the dowel-bar a turn after the concrete has set, or by coating or wrapping the bar with a layer of suitable material. The former method can be used only where the slabs are concreted by alternate bays; the latter method is therefore more generally used, but the layer of material must be very thin if it is not to reduce the efficiency of load-transference, and it is also desirable if it can protect the bar against corrosion across the joint gap. It is also important that the material should not deteriorate with age, in such a way that the dowel-bar becomes locked or loose in the concrete. A thin film of bituminous material, evenly applied, is probably the most suitable to use. Bitumens which can be applied cold are preferable since they can be handled more easily on the site; hot-applied bitumens have the disadvantage that unless the dowel-bars are pre-heated, the bitumen will cool rapidly while being applied and an uneven layer will result. A suitable cold-applied material is a thin bituminous paint consisting of a 200-penetration bitumen, cut back with a suitable volatile solvent.

Other materials have been used, such as grease, waterproof paper wrappings, and cardboard sleeves. Grease is satisfactory if it can be applied thinly and uniformly, and waterproof paper is also satisfactory if it is wrapped very tightly in a thin layer. Loosely wrapped waterproof paper and cardboard sleeves are not satisfactory. Even if the cardboard sleeve initially fits tightly it will compress and deteriorate and in both cases a loose fit will result. As a result of this looseness, impact or hammering will occur during the passage of traffic which will wear or even spall the concrete, and may eventually produce a funnel-shaped mouth around the dowel at the face of the joint. As a dowel-bar in such a condition is useless and repair is almost impossible, great care should be taken to avoid the occurrence of this trouble.

Space must be provided at the end of the sliding half of the dowel-bar to allow for closure of the joint. With alternate-bay construction such space may be provided by pulling the bar out slightly after the first slab has been concreted; but with continuous construction either a compressible pad must be fixed to the dowel or, as is more usual, a cap provided. Such caps may be of cardboard or pressed metal and should be a close sliding fit on the bar. The cap must be set to allow sufficient space for closure of the joint and it should be filled with a wad of cotton waste and the edge sealed during the painting of the bar, in order to prevent it from becoming filled with mortar.

Accurate alignment of the dowel-bars is most important, and in the construction of dowel-bar joints the dowel-bars should be placed parallel to each other and parallel to the surface and centre line of the slab. Failure to do this may result in very high pressures being set up between the dowel-bars and the concrete, either by a wedging action if the dowel-bars are not parallel between themselves or by a lifting or sideways movement of the slabs if the dowel-bars are not parallel to the surface or centre line of the slab. The shear forces produced in the dowel-bars by errors in alignment are given in Table 6 and it is suggested that the maximum permissible tolerance should be ± 0·10 inch in 12 inches for dowel-bars up to and including $\frac{3}{4}$-inch diameter and ± 0·05 inch in 12 inches for dowel-bars of 1-inch and greater diameters. For dowel-bars of very large diameter it would be desirable to reduce the tolerance still further but it is doubtful whether it would be possible to achieve greater accuracy under site conditions.

TABLE 6.—ESTIMATE OF THE SHEAR FORCE IN A DOWEL-BAR DURING MOVEMENT OF A JOINT WITH INACCURATELY ALIGNED DOWEL-BARS. (CALCULATED FOR A CLOSURE OF THE JOINT FROM $\frac{3}{4}$ TO $\frac{1}{2}$ INCH WIDTH)

Error in alignment : inch per 12-inch length	Shearing force : lb. per dowel bar			
	Diameter of dowel-bar : inch			
	$\frac{1}{2}$	$\frac{3}{4}$	1	$1\frac{1}{4}$
0·02	40	100	200	300
0·05	110	260	500	830
0·10	220	500	1,010	1,660
0·25	550	1,280	2,520	4,150
0·50	1,090	2,580	5,050	8,300

Accurate alignment of the dowel-bars can best be obtained by using an assembly which can be cast into the concrete, or by means of drilled and split forms which hold the dowel-bars in position while one half of the joint is concreted. One of these methods should be used because if the dowel-bars are badly out of alignment, not only will large shear forces and consequent high pressure be set up at the joint face but longitudinal movement of the slabs may be restricted.

The concrete around the dowel-bars must be thoroughly compacted because the effective operation of the joint depends entirely upon the bars being surrounded by concrete of high strength. The presence of the dowel-bars and jointing material presents a large surface area against which the concrete has to be compacted, and this large area increases the risk of honeycombing. With a properly designed mix, suited to the type of compactor being used, no difficulty should be encountered in obtaining good compaction, but careful attention to this point is needed.

The success of dowel-bar joints will depend not only upon proper design but also very largely upon careful workmanship in construction; provided the joints are properly constructed initially, no trouble should be experienced for many years and little maintenance will be needed. Faulty construction, on the other hand, may result in the rapid appearance of defects which are difficult and costly, if not impossible, to remedy and in some such cases the dowel-bars may even do more harm than good. The importance of careful construction and proper design of the joint, therefore, cannot be emphasized too strongly.

CONCLUSIONS

(1) The need for strengthening the edges and corners of concrete pavement slabs has been well established from experiment, theory, and practical experience.

(2) Dowel-bars provide the most generally satisfactory method of strengthening the slab edges at joints.

(3) Although solid round dowel-bars are most commonly used there is some advantage to be gained by using tubular bars.

(4) The size and spacing of dowel-bars needed for effective strengthening of the joint under various conditions can be calculated from the results of laboratory tests; results of these calculations are given in the Paper.

(5) Where concrete pavements are laid on very rigid bases, the value of dowel-bar joints is somewhat reduced.

(6) Dowel-bars are effective in reducing the magnitude of the deflexions resulting from loads at joints when the slabs are warped by temperature gradients so that the slab surface is concave.

(7) It is important that the dowel-bars be accurately aligned and free to slide as the joint opens and closes.

ACKNOWLEDGEMENTS

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The Paper is accompanied by three photographs and fifteen sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

The Author introduced the Paper with the aid of a series of lantern slides.

Mr F. N. Sparkes said that the Paper gave much food for thought and in his view was a clearly written, carefully considered assessment of the advantages and disadvantages of dowel-bars in concrete pavements. The Paper was partly theoretical and partly concerned with the results of laboratory experiments supplemented by measurements of joint movements in the field ; it summarized a large amount of data and of observation. He felt it might be useful, however, to look a little more closely into the inferences arising from the work and, if possible, to set a background against which the practical application of the results might best be considered.

First, it must be emphasized that the problem of load-transfer efficiency at joints had been considered in the Paper largely from the aspect of concrete roads. That was perhaps inevitable, because dowel-bars were not normally used in runway construction in the United Kingdom, and it had been necessary, therefore, to use the available information on roads.

It might be argued that because cracking and failures were rare on runways in Great Britain, then there was no problem. Mr Sparkes did not think that was necessarily so. It had been pointed out in the Paper that under certain conditions a significant decrease might follow in the stress intensity at the corners and edges of slabs when dowel-bars were included. It seemed, therefore, worth while to examine whether that decrease in stress could be used to effect ultimate economy in the cost of building, or in the maintenance of a runway, or both. Mr Sparkes said he wanted to underline the need to give consideration to the possibility of effecting economy in the preparation of foundations for concrete runways.

It was common practice in Great Britain to take a great deal of care in preparing a base which was very strong and very rigid, and to attempt to obtain a high modulus of subgrade reaction; he understood that a modulus of as high as 1,000 lb. per square inch per inch had sometimes been used. That was an extremely high value and, with the many types of soils involved, it would be difficult to get that very high value, or even values approaching it, without spending a great deal of time and effort in preparation.

In his opinion the function of such a rigid foundation was presumably to restrict the deflexion at the edges and corners of slabs when loads passed over them. It seemed to him that Mr Loe had shown that the use of dowels might effect a similar reduction. If, therefore, a somewhat lower standard in the preparation, and a lower standard of stability and strength in the foundation were accepted, then the inclusion of dowels might result in no greater stress arising in the slab.

If slabs were designed of uniform thickness, then that thickness should be capable of accommodating the stresses imposed at the corners and edges, and since those were maximum stresses, then the interior part of the slab would be thicker than necessary. That method was normally rejected by airport engineers as being impracticable. It might be possible to use slabs of different thicknesses—a few inches thick in the central area and thickened at all the edges—but the economies arising from the use of a smaller thickness over the interior portion of the slab would have to be balanced against the increased cost of preparing a shaped formation.

The airport engineer would want practical evidence to show that an economy or a better performance was likely to be achieved by either one or both of those methods. He had very little long-term experience to guide him in his designs and, quite naturally, he would be anxious to know whether or not they had passed laboratory tests. Mr Sparkes therefore strongly supported Mr Loe's suggestion that a series of full-scale accelerated traffic tests on specific items should be arranged, to investigate such questions as to whether or not economy could be effected, or whether the stress distribution could be made more uniform by the use of dowel-bars. The relative advantages of, and possible economies by, the use of dowel-bars could then be readily stated in a way which would be acceptable to the airport engineer. He was very glad that some such arrangements had been made and he felt that they would produce most valuable results.

Mr A. F. Lambert said that the Paper had a special interest to him because it had a bearing on a question which he had asked 3 or 4 years previously at an informal meeting of the Airport Division. On that occasion a Paper was not being discussed but, instead, three gentlemen from the Air Ministry sat at the high table and constituted a sort of technical brains trust. Members of the audience asked questions, and each member of the "trust" replied in turn.

Mr Lambert had been personally engaged at that time in the construction of the de Havilland runway at Hatfield, which was the only fully dowelled runway in Great Britain; he had asked whether the fact that the Air Ministry did not use dowel-bars or other load-transfer devices resulted from their being technically unsatisfactory, or because they were too difficult (from a practical point of view) to build into the slabs. The reply had been slightly evasive, but it had indicated that the Air Ministry did not consider that the case for dowel-bars was fully proved, and until it was fully proved, it was not proposed to use them.

Mr Lambert said that the Paper had gone a long way to fill the gap. The Author had pointed out the different circumstances in which it appeared that dowel-bars might help to solve the problem and had admitted that there were other circumstances in which the case for them was far less certain. That was the type of Paper—unprejudiced—which had a very special value.

Mr Lambert said that there were several references to longitudinal joints in the Paper, but he was left with the impression that the Author had been considering only transverse joints and, in fact, transverse joints where the dowel was required to slide. The Author had defined dowel-bars (on p. 614) as bars which allowed sliding, but Mr Lambert said he was not quite prepared to accept that definition. At Hatfield they were called dowel-bars when they held together the two sides of pre-formed joints, and they were called tie-bars when they were used for holding together the cracked faces which were expected to develop at a dummy joint. Would the Author state whether he had in mind dowel-bars or tie-bars, call them what he wished, in longitudinal joints? It seemed logical that if the conclusion was that they were required transversely, then they must also be required longitudinally. In fact, on p. 615 the Author quoted from Westergaard in which it was shown that for a certain assumed load in a certain hypothetical condition the stress in the longitudinal joint was slightly higher than the stress in the transverse joint.

The Author had compared the advantages of strengthening the slab by the use of dowel-bars, with thickening the slab. In his calculations he had said: "this cost is given in shillings per square yard of concrete for a case where the length of joint amounts to 150 feet per 100 square yards of concrete (that is, for 12-foot-square slabs)." That would appear to provide for a joint on only two sides of the 12-foot-square slab and confirmed Mr Lambert's impression that the Author was thinking all the time of transverse joints. If that were so, and if the Author agreed that logically some form of dowel or tie was necessary longitudinally as well as transversely then the cost allowed for dowel-bars in the example would have to be doubled. In that case, Mr Lambert suggested, the dowel-bar would not appear in so favourable a light when compared with thickening the slab.

The next point on which he commented was in connexion with the apparatus shown in *Figs 6* which was used in the laboratory for testing individual dowel-bars. On p. 627 the Author had commented: "With this method of testing, that part of a wheel load which would normally be counter-balanced by the reaction of the subgrade under the loaded slab is eliminated." The inference was that the dowel-bar was being more severely tested in the apparatus than it would be in practice. Mr Lambert agreed, but it seemed to him that the apparatus failed to simulate practical conditions in another respect. The slab shown as 42 inches long could be considered as being in a semi-fixed condition at the dowel-bar end, but supported on a roller bearing and therefore pin jointed at the other end. In practice, the slab had a degree of fixity at the dowel-bar end, but there was no other clearly defined end to the slab. Certainly it was not pin jointed. Some distance back from the joint, reversal of stress must take place, tension would develop in the upper part of the slab, and the flexural stiffness of the slab would come into play and tend to relieve the dowel-bar of the load to which it would be subjected in the apparatus. If he were correct, the error, if it was an error, was on the right side, and it simply meant the dowel-bars tested in the apparatus were subjected to a more severe load than would be the case in practice.

Lastly, Mr Lambert said that he was strongly in favour of solid round bars, as distinct from tubular bars, in view of the risk of corrosion developing on the inside of the tube. The Author had carried out his tests on solid round bars, but at the bottom of p. 629 had said that tubular bars, or square or rectangular section, could be used, and he had made out a case for greater rigidity of those sections for the same weight of steel. Probably that was so, but it was always assuming that the dowel-bar was firmly embedded in the concrete. He was sure the Author appreciated, as some parts of the Paper indicated, the need for first-class workmanship in the use of dowel-bars—the whole principle of the dowel-bar depended on its being very well built-in.

At Hatfield, round bars 1¼ inch diameter had been used, and in more than 99 per cent of the cases the dowel-bars had been firmly held in the concrete; but there was the odd case in which a small cavity had formed underneath the dowel-bar, showing that, owing to some local segregation of the concrete, the concrete had moved downwards under the influence of the vibrating machine but had not moved sufficiently under the bar to provide a proper bond. Very occasionally a bar had been found so badly held that it could be turned by hand. If that happened with round bars, he thought it would happen far more with square or rectangular bars.

Mr Lambert then showed some slides illustrating the construction of the slabs and joints used in the runway at Hatfield. He observed that once a satisfactory technique for holding the bars in the formwork had been evolved, and the workmen had become trained in the setting up of the jigs, etc., the incorporation of dowel-bars in a runway slab was not a very serious problem, and those practical matters should not prevent dowel-bars from being considered in any circumstances where they appeared to be satisfactory from theoretical considerations of the design.

Mr G. S. Cooper said that the Paper had shed some light on one of the most debatable subjects encountered in pavement design. Many different opinions had been expressed on the degree of stress relief given by the use of dowels. Mr Snow, in his Paper on the de Havilland Airfield,[1] had stated that edge stresses were reduced by as much as 45 per cent and Mr Loe stated that 40 per cent relief should normally be obtained. On the other hand, the Corps of Engineers, U.S. Army, with probably more experience of designing, constructing, and testing airfield pavements than any other single authority, had assessed the relief at 25 per cent. Mr Cooper quoted from the Corps of Engineers' Rigid Pavement Design Manual, which had been issued in 1951 : " The important function of dowels or any other load-transfer device in concrete pavements is, first, to maintain the alignment of adjoining slabs and, secondarily, if possible, to limit or reduce stresses resulting from loads on the pavement. Experience has shown that by the use of conventional steel dowels, also some of the proprietary devices, the first of these objectives is easily obtainable. On the other hand, it is significant that pavement condition surveys, extensive laboratory testing and various attempted theoretical analyses have thus far failed to establish a true measure of the effects of the dowels on stresses in the pavement. It is not unlikely that ' load transfer ' under certain conditions, as uneven support of the pavement by the subgrade, or where any of the slabs are weak or broken, results in increasing maximum stress rather than reducing it."

On the economic side of the question, the Author had tried to show that a saving of about 2 shillings per square yard could be obtained by using dowels in a pavement of a given thickness. However, the 20-foot-square slabs, in the experience of the Air Ministry, usually tended to quarter. That could often happen when the slabs were designed for corner-stressing conditions under the worst conditions of night stress. Therefore, since there were not many 15-foot machines available, 10-foot-square panels, with a central joint, had been adopted.

If the Author's 20-foot-square panels had cracked transversely and longitudinally, then there would be three undowelled joints in the middle and the advantage of the original dowels would be nullified. If, however, the dowels were put in at 10-foot centres, the cost in the example was doubled and there was no economic gain. There was also the question of steel supply. That was a very difficult matter, and all alternatives, even though slightly more costly, such as adding extra concrete, must be carefully considered. The Author was optimistic about steel becoming plentiful again, but it should be borne in mind that 1,300 tons of steel were required to provide dowels in the present-day single-runway airfield for heavy aircraft, and Mr Cooper thought it might be many years before that amount of steel could be used without any qualms.

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[1] F. S. Snow, "The de Havilland Airfield, Hatfield." Instn Civ. Engrs, Airport Paper No. 12, 1949.

No reference had been made by the Author or by later speakers to the performance of dowels. So far as he knew, there was very little evidence available to show how dowel-bars had performed after, say, 5 years. It would be very interesting if some could be taken out at the de Havilland airfield to see whether they were, in fact, still functioning as dowels. If they were free and able to slide in their sockets, or, alternatively, if they had been moving and had worked sufficiently loose to allow $\frac{1}{8}$ inch play (this had actually occurred in America), then they were quite useless.

Mr Cooper felt that despite the advantages—lining up the slabs and limiting deflexion—the case for dowels had still to be proved.

Mr B. J. F. Bradbeer said that, having been engaged for more than 25 years in the construction of concrete roads, it had been a pleasure to read the Paper and to see in it the elucidation by instrumental measurement and scientific calculation of many of the problems with which engineers had been confronted in past years when, without such technical appreciation, decisions had had to be taken about the use of dowels and tie-bars based only on practical experience, very simple calculations, and on positive, observed results. During the 10 years prior to 1939 he had taken an active part in the construction of about 50 miles of concrete carriageways in one county in Great Britain, and all of those roads were to-day in substantially good order.

It might be interesting to reflect that in those roads dowel-bars were introduced tentatively in 1931, in embankments and doubtful ground. Some tie-bars—across the longitudinal joints—were also used at that date. By 1935 the evidence appeared to show that both the dowels and the tie-bars offered an effective answer to many of the problems. From 1935 onwards all the roads—that was, the greater part of the 50 miles—were constructed with dowels and tie-bars. One of the first results of the dowel-bars was a considerable saving on maintenance of the joints and an almost complete disappearance of deflexion and spalling of the joints. In the case of the tie-bars, the parting of slabs, which had occurred here and there before, appeared to be mastered. The cost (part of a penny per square yard of a road) had been considered to be worth while as a form of insurance.

During the war, maintenance of the joints was sadly neglected, and it was interesting to find that although many of the roads had been under unusual traffic in anti-invasion and other training exercises, with heavy vehicles and tanks, the dowelled and tied joints had suffered very little damage and the results were satisfactory. In all cases the dowels were of smooth round mild steel, $\frac{3}{4}$ inch or 1 inch diameter, at 12- to 15-inch centres according to the slab thickness and estimated weight of traffic. Great care had been taken to set the dowels parallel, for which reason the length of the dowels was not less than 3 feet to enable them to be held in some form of jig during concreting. The dowels had been made to slide freely at only one end by a wrapping of waterproof paper, with a cap in most cases, and the only indication of possible improvement in those early days, was that the paper wrappings were too thick. In some of the older roads there was some evidence of movement and a slight bumping could be heard during the passage of very heavy lorries; that had been corrected in the later design. Quoting from his own experiences in connexion with one particular road, Mr Bradbeer said that for several miles the sliding had been provided for by the use of thin metal sleeves, but after several years there was evidence that in some cases they had rusted, so preventing sliding, because nowadays the expansion and contraction sometimes took place over as many as three bays together—150 feet. That might be an advantage; it had been learned that joints could be made farther apart than they were in those early days. Dummy joints had been employed in places and had been tied with indented bars, some of which, 18 years old, had held the faces in close contact and had prevented the entry of any foreign material. The joints were quite free from any form of spalling.

Tie-bars across one or two joints were of hooked or hair-pin shape, of various sizes and spacings, but there was always provided a minimum of one square inch cross-section of steel for every 10-foot-run of joint. That had been completely effective in preventing the parting of slabs which had been seen on some of the older roads. That parting appeared to occur on roads where the foundations at the edges were likely to be affected by changes in moisture content during the variation of the seasons. The edges of the slab in bad soils tended to rise and fall according to the extremes of drought or heavy rainfall. In the worst case encountered, the channels of the 20-foot carriageway had risen and fallen 2 inches, and the complete range of movement in one season had taken place in a period of only 7 weeks. It appeared that when the edges of the road were depressed, the centre joints, unless they were well tied, opened and permitted foreign matter to enter the top part of the joint. When the edges of the road rose there was a nut-cracker effect, and the very great leverage could force the slabs apart. Unfortunately, that tended to be a cumulative movement where ground conditions were of that unfortunate nature. The horizontal force necessary to move the slab sideways did not appear to be great, from Mr Bradbeer's observation, because he had found it possible to check the movement very simply by the use of large mild-steel dogs, about 5 feet long with hooks at the end. Drilled into the concrete, they had been quite effective, and the stress did not seem to be large, but the little increments each season did have the effect he had indicated unless bars were in place. On the worst foundations, even where the bars were in place, there had been some movement.

In that respect, so far as the modulus reaction of the clay upon which many of those roads were built was concerned, a modulus of subgrade reaction as uniform as possible at 200 to 250 would normally be accepted, as compared with 1,000 mentioned in the Paper. Probably that was why particular attention had been paid to that class of work in East Sussex. Since 1935, he estimated, about 120,000 of those bars had been used and in no case had there been any sign of failure or cracking away of the bar, or of breaking of the concrete caused by local excessive stress.

Mr W. P. Andrews complimented the Author on the Paper because it covered a subject on which far too little had been written in Great Britain, but on which there had been a lot of research in the United States. Discussing the subject with engineers there, he had been surprised at the meticulous care taken both with the workmanship and with the specifications. The State of New Jersey, for example, had a specification for dowel-bars and assemblies running from two to eight pages compared with our five or six lines. It was interesting to recall that as long ago as 1934 in that State it had been found that $\frac{3}{4}$-inch dowel-bars were not satisfactory—they rusted and broke—and after 1934 a standard $1\frac{1}{4}$-inch bar had been adopted. The Chief Engineer, Mr Van Breemen, had told Mr Andrews that his greatest problem with dowel-bars had been to ensure the prevention of bond between the concrete and the bar. In the course of time a most complicated arrangement had been evolved in which the dowel-bar was enclosed in a shaft of stainless steel which fitted tightly over the bar; the inside of the shaft was given a slight coating of oil. The shaft covered the free end of the bar and extended through the joint into the other slab, so that there was no corrosion at the joint.

It was also interesting to note that in the same State, which was one of the most progressive in the United States, most unsatisfactory behaviour by the cills or sleepers under the joints had been observed. Their cill was most elaborate—5 feet long, 3 inches thick at the edges, tapering to 7 inches at the centre, and reinforced. Under heavy traffic, even in reasonable subgrades, the cill was driven down into the subgrade and gave entirely unsatisfactory service.

Reference was made on p. 616 to tongue-and-groove joints. Mr Andrews had always wondered why it was that they made their tongue-and-groove with a slope on the tongue. It might be for easier construction, but it meant that if the slabs moved apart, one went downwards and it was more difficult for the slab to get back and close the joint again. A more sensible arrangement was the interlocking joint (illustrated by a slide) designed by the late Mr J. H. Walker, a Member of the Institution. It was well known to the Air Ministry and had been used extensively during the war.

Mr Andrews referred to a joint he saw about 20 years ago in Lancashire. It consisted of three steps in a 10-inch-thick slab (4 inches, 3 inches, and 3 inches) with the treads covered with bitumen, which allowed one slab to move on the other. It had the great advantage of being practically water-tight since it stopped water from getting through the joint into the subgrade. In the risers the expansion joint filler was incorporated. Mr Andrews said he would have expected cracking to occur, but in fact after 20 years the joints were still perfect.

He endorsed Mr Loe's remarks about the necessity for good workmanship, the smoothness of the bars and their freedom from burrs at the ends, their true alignment, and the provision of caps to permit the movement of slabs. Reference had been made to assembly units and the absence of really satisfactory units on the market. Mr Andrews had seen in Washington and Chicago, museums of assembly units—museums of failures—and had been told by leading engineers of the Federal Government and other prominent engineers that there was not one satisfactory assembly unit on the market. He had also found that the cost of an assembly unit was very high.

Mr Andrews thought the best thing was to improvise. He showed a slide to illustrate that dowel-bars could be effectively positioned by timber or steel forms. He said that in his experience, lack of alignment in all planes had prevented the expansion joint from functioning and had caused cracks to develop immediately above the end of the bars about 18 inches from the joint.

Mr Andrews referred to a pre-war case of a cycle track where dowel-bars had been put in but with no caps on the end of the bars and no provision for expansion. The work had been done in cold weather, and in hot weather the slabs expanded and one slab rose about an inch higher than the adjoining slab, bending the bar, and thus preventing the slab from returning to its original position. The engineer concerned had condemned the principle, but it was not the principle which was wrong—it was the failure to provide for expansion.

Mr Bradbeer had referred to dowel-bars used in East Sussex 20 years ago. The earliest use of dowel-bars to his knowledge, Mr Andrews commented, was in 1927 in a road constructed over a peat bog. The road was 9 inches thick and doubly reinforced, in which dowel-bars 3 feet 6 inches long had been used. That road was still in perfect condition to-day; there was not a crack. However, in the course of time the dowel had worn away the concrete above it and, although the road surface was perfectly good, when a heavy lorry passed over the road there was a " bump, bump " at each joint. One speaker had suggested that there was no experience with dowel-bars 5 years old, but those were 25 years old. The result was perfectly satisfactory except for the " funnelling," and Mr Andrews was not sure what the cure for that was. The only solution appeared to be to put another 6-inch reinforced slab on top, breaking joint with the present joints.

Mr Andrews exhibited some slides illustrating the positioning of dowel-bars, under different conditions, and showing examples of bad alignment in two cases and careful workmanship in another.

Mr N. F. Truscott said that there seemed to be a slight discrepancy between *Figs 14* and *17*. In *Figs 14*, in the case of a loaded lorry (*c*), there appeared to be a range of relative deflexion of ·075 inch, under the worst conditions. In *Figs 17*, in the plain slab, which was the comparable case, there was a range of movement of only ·006 inch. So far as he could see, the conditions in the second case were worse than in the first. The first was an 8-inch slab, on a gravel base, the vehicle weighing 16,600 lb.; the second case was a 6-inch slab, on an unspecified base, the vehicle weighing 19,200 lb. Yet there appeared to be one-twelfth of the deflexion compared with that found in what appeared to be a less severe case.

*** **Dr Guthlac Wilson** observed that on p. 621 the Author gave as the third requirement of a dowel-bar joint that the "dowel-bars shall not unduly restrict the freedom of the slabs to warp." It was not clear why that was undesirable. It would appear at first sight that restriction of warping would be helpful, since it would tend to keep corners and edges down to the ground.

In Table 4, on p. 645, the Author gave failing loads of dowel-bars and suggested working loads. Dividing the latter by the former, it would appear that the recommended factor of safety for $\frac{1}{2}$-inch and $\frac{3}{4}$-inch dowel-bars was 2, that for 1-inch dowel-bars was 1·5 and that for $1\frac{1}{4}$-inch was less than 1·3. Why was it recommended that the factor of safety should drop so sharply with increasing diameter of bar? Dr Wilson suggested that a factor of safety of less than 1·5 was undesirable.

The Author, in reply, said that he thought the suggestion put forward by Mr Sparkes that the extra support given by dowel-bars to the edges and corners of slabs would enable a less rigid foundation to be used, was one which would be well worth testing on a full scale. Very rigid bases were both difficult and costly to prepare and their rigidity was not always an advantage; it was believed that the inability of such bases to accommodate themselves to the warping action of slabs did, under some conditions, cause cracking.

Mr Lambert had suggested that dowel- or tie-bars should be used in the longitudinal joints, and the Author fully agreed with that suggestion. In *Figs 3* allowance had been made for dowel bars in all joints; it was based on the case of a slab which was surrounded by similar slabs, and where there would be an average of one transverse and one longitudinal joint per slab.

The method of test was designed to simulate the conditions to which a dowel-bar was subjected in a road slab. The load applied to the test specimen was equivalent to the load transferred by a dowel-bar in a road joint, and the roller bearing used in the test specimen allowed some angular movement to occur at the same time as the vertical displacement. The Author agreed with Mr Lambert that road conditions were not reproduced exactly but that could be done only by very large specimens—unsuitable for laboratory tests—or by the use of a complicated mechanism which would reproduce the movements due to warping.

It was very interesting to hear from Mr Lambert that experience in the construction of the de Havilland runway at Hatfield had shown that,

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*** This contribution was submitted in writing.—SEC. I.C.E.

with a satisfactory technique and trained workmen, the installation of dowel-bars was not a very serious problem.

Mr Cooper had quoted from the Specification of the U.S. Army Corps of Engineers, in which it was stated that the function of dowel-bars was to maintain the alignment of adjacent slabs, and that the limitation or reduction of stress was a secondary matter. Even so, they assessed the stress reduction at 25 per cent and evidently considered that worth while for they still continued to specify dowel-bars and were using bars of 2¼ inches diameter in certain cases. The Author's experience with the thinner slabs and lighter loads on roads had indicated that the stress reduction in such cases was greater than 25 per cent. Mr Cooper had said that the question of using dowel-bars was the most debatable subject in the design of pavements for airfields and that the case for dowel-bars had still to be proved. The Author thought that clearly indicated a very real need for tests to be made on airfield slabs in Great Britain, where conditions often differed appreciably from those in America. Particular attention should be paid to the performance of joints under night-time conditions; the importance of that was clearly shown by *Figs 14*, *15*, *17* and *18*.

Mr Cooper reported that the 20-foot-square slabs tended to quarter and that it had been necessary to sub-divide these slabs into 10-foot-square bays, which of course would mean having a very large quantity of dowel-bars. The Author felt it was rather a pity that machines could not be built or modified for constructing 15-foot-wide slabs; experience in Great Britain and in America had shown that the joints could be spaced at 15-foot intervals with only a very small risk of the slabs cracking. Compared with 15-foot-square slabs, 10-foot-square slabs needed 50 per cent more joints and it seemed that much extra maintenance was being created, to say nothing of the cost and the problems of forming the additional joints in the first instance.

Mr Cooper had asked how dowel-bars performed after, sat 5 years. Since the Paper was written it had been possible to make some measurements on a road which had been constructed in 1931, and had carried constructional traffic for 2 years and a large volume of heavy traffic ever since. The road contained both plain and dowel-bar joints. Measurements revealed that horizontal movements were occurring in all joints but that the amount of movement at the dowel-bar joints was a little less than at the plain joints. That showed that, with proper construction, dowel-bars did not become completely bonded into the concrete after a period of time. The relative vertical movements at the dowel-bar joints did not exceed 0·001 inch whereas at the plain butt joints it averaged 0·059 inch. That showed that the dowel-bars remained effective as load-transfer devices.

Mr Bradbeer's experience had demonstrated, in a practical manner, the value of dowel-bars in joints in roads. The question of maintenance was extremely important, and anything which could be done in designing a runway to reduce maintenance or avoid undue risk to the pavement, if maintenance had to be neglected at some time, should certainly be fully considered.

The Author was glad to hear Mr Andrews's remarks on the need for good workmanship and accurate alignment of the dowel-bars. His experience that it was best to improvise supports to position the dowel-bars, using timber or steel forms, was a point to be noted.

Mr Truscott had referred to the difference between *Figs 14* and *Figs 17*. The scales were correct but the Author was glad Mr Truscott had drawn attention to that, since the records given in each Figure were intended for comparison within themselves and not between the Figures. The measurements had been made under different conditions, and three factors would contribute to the differences in the deflexions. First, the apparatus had been placed close to the track of the vehicle for the measurements in *Figs 14* whereas it had had to be placed 4 feet away for those in *Figs 17*. Secondly, the vehicle used for the measurements in *Figs 17* had had its weight nearly evenly distributed between front and rear axles, the rear-axle load being 10,300 lb. compared with 13,500 lb. for *Figs 14*. Thirdly, *Figs 14* and *Figs 17* (*a*) referred to plain butt-joints where the amount of load-transference depended upon whether or not the slabs were butted together at the joints. In *Figs 17* the slabs had been more closely butted together at the time of making the measurements than in *Figs 14*.

Dr Guthlac Wilson had asked why it was necessary not to restrict the warping action of the slabs: that was because, when a slab was subjected to temperature gradients, high stresses would develop unless it were permitted some freedom to warp by the flexibility of the joints. It was desirable therefore, to permit warping to take place, although it did result in increased relative movements at the joints.

Regarding the factor of safety, it was not felt essential that that should be large, for the reasons stated in the Paper, and it was considered that a small margin between working stress and ultimate strength, rather than a percentage factor of safety, would be adequate. The Author realized that that might be open to question but he felt that the values suggested were a reasonable compromise between providing some safety margin, and the need for economy in steel.

Finally, the Author hoped that the Paper would lead to further consideration being given to the problem of whether or not dowel-bars should be used in airfield construction; he thought that there was sufficient evidence in their favour to justify the making of full-scale tests. The discussion had shown that dowel-bars in roads had resulted in worth-while benefits, and that their use in at least one airfield had not led to undue difficulties in construction.

Correspondence on this Paper is now closed and no further contributions may be accepted.—SEC. I.C.E.

ROAD ENGINEERING DIVISION MEETING

29 April, 1952

Brigadier A. C. HUGHES, C.B.E., T.D., B.Sc.(Eng.), M.I.C.E.,
Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors

Road Paper No. 38

"Mechanical Surfacing of Roads"

by

Stuart Maynard Lovell, O.B.E., T.D., A.M.I.C.E., Edward William Walter Richards, A.M.I.C.E. and Thomas Urie Wilson, C.B.E., T.D., D.L., M.I.C.E.

SYNOPSIS

The Paper first describes the preparation of the road surface by means of th method of heating and planing. The principles underlying this work are described suitable conditions for the method are detailed; the types of machinery available the results obtained, and the operating data, including typical costs, are given. Th dangers inherent in this method are also described.

The first method of producing the finished surface to a road is dealt with unde "Non-Power Operated Spreaders." The type of machinery available, with a description of its construction, its method of operation, data on its use, and results obtaine are fully described. The limitations of this type of machine are also dealt with.

Power-operated machines in common use are treated at greater length and the Paper includes a detailed description of the Barber-Greene machine, the history of it advent into current practice, and a full account of the principles on which it operates A critical analysis of the economic considerations lying behind the efficient use o this machine is given, supported by current American thought and practice on thi matter and illustrated by means of statistical data. Two charts are included, on illustrating the economics of the supply of materials to the machine, the other giving faults occurring in operation and their remedies. A code of practice for the best use of the machine is included.

The latest development in the control of the supply of materials by radio to a Barber-Greene machine is described under the heading "Intercommunication."

At the end of the Paper are some practical data for adequate maintenance of the Barber-Greene machine.

INTRODUCTION

THE Committee's terms of reference for this Paper were purposely and strictly limited. It was not to be a treatise on mechanized road-making but was to deal solely with the use of machines for the production of the finished running surface (excluding concrete). At the same time, the

Committee asked that whilst the mechanical resurfacing of roads and the various machines available and their methods of use should be covered as widely as possible, the Paper should also deal with the running maintenance of the Barber-Greene machine, because this type of machine was the one most generally used for producing finished surfaces. Consequently the machines and methods for heating and planing, and for non-power and power-operated spreaders, constitute the scope of the Paper. Lack of space prevents any reference to mix-in-place machines such as the grader, the Woods Machine, P. & H. Stabilizer, and other machines of American origin.

It will be seen, therefore, that the Paper has limitations set by the terms of reference. There is no mention of any mechanized work on foundations, on drainage, or on what might generally be described as major shaping-up operations such as superelevation, except those that can be executed by the Barber-Greene. Neither does it cover the present practice in Great Britain for the manufacture of materials for spreading by mechanical means.

HEATING AND PLANING

Hitherto the elimination of corrugated surfaces has presented a difficult problem, because surface dressing, and even resurfacing, has often resulted in the new surface assuming the irregularities of the old. Moreover, the continued impact and vibration of traffic on such corrugated surfaces may seriously threaten the stability of the carriageway foundations, apart from causing considerable discomfort to the drivers and passengers of all classes of vehicles.

The development of combined heating and planing provides an aspect of road maintenance whereby corrugations and similar malformations can be removed and a reasonably true running surface obtained, without disturbing or injuring the carriageway foundations. Where applicable, this method of treatment is particularly advantageous in cases where it is impracticable to raise the level of the carriageway in urban areas by reason of the levels of adjoining footpaths and properties. It is also suitable for the removal of old surface dressings which, owing to excessive binder content resulting from repeated surface dressing applications, have become corrugated with consequent irregular riding surfaces, in addition to the removal of asphalt surfaces preparatory to resurfacing. Furthermore, the cost of laying an asphalt or carpet on a true surface is much less than on an uneven surface, owing to the greater covering capacity per ton of material. The planed-off material which is removed by this method can be re-used, while still hot, for the surfacing of secondary roads and footpaths, but it frequently needs some subsequent surface dressing treatment, owing to the effect of heat on the original binder.

Three types of heating and planing machines are available which are generally of similar features though differing in the method of planing. Normally a battery of two or three heaters is used, each heater being about 7 feet square and so spaced that the heating is progressive. Following the first application of heat, the hot combustible gases are dissipated before the second application, and again before the third application, so that the ignition of these gases and consequent charring of the road surface is thus avoided. The penetration of heat through the road surface is achieved by hot air and by direct radiation within the heated hood, so that the flame does not come in direct contact with the road surface. On very "fatty" surfaces, with a low flash point, some temporary ignition of these gases may be experienced, but since it is only vapour which so ignites, it does not affect the value of the material.

The heating and planing unit is usually designed with as long a wheel base as practical considerations will allow and the rotary or diagonal cutting blades are set to produce the maximum degree of planing possible, whilst a positive adjustment for depth of cut is provided and blades can be cambered as required. In earlier experimental machines, it was discovered that fixed planing blades were not entirely satisfactory; that resulted in the introduction, in some machines, of reciprocating blades which move transversely to the direction of travel of the machine.

In practice, it has been found that at least two light cuts are preferable and more effective than one deep cut, whilst subsequent cuts can be undertaken to reduce excessive camber. In exceptionally severe conditions a system of scarifying tines can be brought into operation in front of the cutting blades to loosen up the heated material on a deep cut before planing. Depth of cut can be varied according to conditions from ¼ inch to 1½ inch on surfaces ranging from surface dressing to stone-filled asphalt.

The planed-off material or windrow is discharged centrally at the rear of the machine and is easily recoverable either by hand or by mechanical means. The rate of progress of the machine generally varies according to the type of surface and depth of surface to be removed. It has a capacity of about 2,000 square yards per day or 12,000 square yards per working week when making two cuts on ordinary surface dressings; the progress is correspondingly slower on asphalt surfaces or when a particularly deep cut is required.

The costs of the various kinds of work undertaken by such machines are of the order shown in Table 1.

The surface, so heated and planed, generally provides a smooth running surface, since all corrugations and irregularities are thus removed together with all surplus binder content and fatty portions. The surface, however, presents an open and dry texture and benefits by being kept open to traffic for some time before being resurfaced. The wheel action of traffic tends to remove some of the excess of binder brought to the surface by the heating and planing process and by atmospheric action, and, in consequence, the possibility of a new carpet becoming "fatty" is largely

TABLE 1

	Up to 10,000 square yards: cost per sq. yard	10,000 to 20,000 square yards: cost per sq. yard	More than 20,000 square yards: cost per sq. yard
Tar and bituminous sprayed surfaces:			
Up to 2 cuts . .	1*s.* 5½*d.*	1*s.* 4½*d.*	1*s.* 3½*d.*
3rd cut . . .	7*d.*	7*d.*	7*d.*
Bituminous carpets:			
Up to 2 cuts .	1*s.* 8*d.*	1*s.* 7*d.*	1*s.* 6d.
3rd cut . . .	9*d.*	9*d.*	9*d.*
Cold asphalt:			
Up to 2 cuts .	1*s.* 9*d.*	1*s.* 8*d.*	1*s.* 7*d.*
3rd cut . . .	10*d.*	10*d.*	10*d.*
Hot asphalt:			
Up to 2 cuts .	2*s.* 0*d.*	1*s.* 11*d.*	1*s.* 10*d.*
3rd cut . .	11*d.*	11*d.*	11*d.*

minimized by adopting this course. The heated and planed surface should, however, be either surface-dressed or resurfaced within a few months to obviate deterioration under traffic and weather conditions.

NON-POWER-OPERATED SPREADERS

The non-power-operated spreader has been designed, generally speaking, for spreading the finer dry and coated materials, although it can be used, under favourable conditions, in laying the coarser grades. Its object is to spread such road materials to uniform pre-determined thicknesses at a more rapid rate and to a higher standard of grade level and texture finish than can be accomplished by skilled hand labour. This is achieved by a screeding action in which the material is struck off by one or more screed blades, which are set to the required thickness of coat in relation to the machine's runners, or skids resting on the prepared base, and on which the machine is propelled. Manually operated adjustment is provided to vary the thickness and width of the applied coat, and the required camber can be pre-set. Being devoid of any means of propulsion, it relies for its prime mover on the vehicle delivering the material, or on some independent towing unit.

There have been two such machines produced in Great Britain:

(*a*) The Rapier Adjustable Spreader and
(*b*) The Phoenix-Edwards Spreader.

The former machine went out of production in 1948, and it is understood that it is not intended to produce any more. It will be necessary, therefore, only to consider the second machine. The Phoenix-Edwards

Spreader was introduced on the market about 2 years ago, and the utmost simplicity has been aimed at in the design of this machine. It is a much lighter machine, weighing only 17 cwt. compared with the 2 tons of the Rapier Spreader, and it is of such simple operation that it is well within the capabilities of the ordinary roadman to use efficiently.

It consists of a rectangular steel framework mounted on two pneumatic-tired retractable wheels on which it forms a two-wheeled trailer, easily manoeuvrable by one man, and, being provided with a rigid towbar, can be towed from job to job by any road vehicle fitted with a tow hook. For operating, the wheels are retracted by means of fitted screw jacks, and the machine then rests on skids or runners. Three V-shaped screeds are fitted, the centre one having its apex to the rear and having bleed gates at the open apex by which an operator, standing on the platform provided, can control the flow of material to the rear finishing screed. The centre and rear screeds are extendible giving minimum and maximum operating widths of 6 and 8 feet respectively. The whole machine can be adjusted vertically from the runners by means of screw jacks, to give a thickness range of from 0 to 3 inches.

In operation the machine is coupled to the delivery lorry by means of chains and the material is fed from the lorry directly on to the road in front of it, either by manual shovelling or by partial tipping of the load, or a combination of both, as the work proceeds. The bleed gates are controlled by an operator standing on the platform provided on the main frame, from which position he controls the flow of material to the rear finishing screed. Little skill is required, and an intelligent roadman can, after a little practice, arrange the flow of material so as to leave a windrow on both sides or either side of the machine for feathering out at the channel or centre joint, or not, as required. A normal team for operating this machine consists of two men feeding material from the towing lorry, one operator, and two or three men with rakes feathering out at the channels or centre joint according to the nature of the road edge. This total of six men can be reduced if the material is fed by tipping from the lorry.

This type of machine is capable of dealing quite effectively with the small-aggregate materials, and is most efficient on fine grades such as cold asphalt. Although capable of handling dry and coated materials up to 1½-inch grading, the machine, being of light construction, is not suitable for coarser material. Its output appears to be limited only by the rate at which the material can be fed to it, and this ranges from 12 to 20 minutes (according to the experience of the gang) for a load of 6 tons. The daily output of the machine, however, is usually reckoned as being 90 to 100 tons, since delays arising from hitching the spreader to each lorry that arrives on the site, adjustment to the machine, and other unavoidable causes may reduce the actual spreading time of the machine by about 2–3 hours.

The finished results are considerably better than those obtained by hand spreading, and output is much increased. Since the machine operates on runners, local depressions, or high spots less than 10 feet long are eliminated, and a very smooth running surface is obtained. Owing to the screeding action in laying, little or no segregation takes place, and the finish presents an even and uniform texture; thus one of the major faults of hand spreading is eliminated.

The non-power-operated spreader cannot compete, in major works with thick applications, with the Barber-Greene Finisher, which, by means of its floating screed, can anticipate and correct high spots and depressions of much greater magnitude and length than are within the capabilities of this machine. Relying, as it does, on the free flow of material against a screed blade, the supply of coated materials must be at temperatures to allow of this free flow, and, except for cold asphalts designed for cold laying, low temperatures of materials and climatic conditions tend to cause dragging, particularly with high-viscosity binders; this is a tendency to which all mechanical spreaders are inclined—although to a lesser degree where the screed can be heated. Generally speaking, the usefulness of the non-power-operated spreader is most evident with the smaller grade materials, and the ease of mobility and its economic adaptability to the smaller jobs makes the Phoenix-Edwards Spreader a useful addition to the plant of a highway authority.

POWER-OPERATED FINISHERS

General Principles

During the past 10 years there has been an almost complete change-over from manual laying of surfacing materials to laying by highly specialized mechanical finishers incorporating automatic arrangements for feeding, distributing, compacting, and striking-off the coated surfacing material, which is fed direct from the delivery lorry into the hopper of the machine. At no stage of this process need the material be touched by hand; the results obtained are incomparably better than those from hand work; and the cost is less because the machine can accept and lay many times more material in a day. The use of these mechanical laying devices began in the United States before the second World War and was first introduced extensively into the countries of the Western hemisphere in the laying of of surfacing materials on airfields. After the end of the war the use of these machines for road making was somewhat tentatively begun and, as experience of their capabilities and satisfaction with their performance become more widespread, their use increased at an accelerated rate.

Before embarking on a consideration of the actual laying of surfacing materials it is important to realize that machine laying and hand laying involve separate and widely different techniques, and that in both conception and practice it is incorrect to regard one as a development of the other. Hand laying is an art, dependent upon an acquired personal skill in the use of hand tools. Little talent for organization is required, the pace of the work is that of the manual labourer, the daily output and expenditure are small, and the results are, by present standards, of indifferent quality. Machine laying, on the other hand, requires a greater knowledge of a wider range of surfacing materials, a skill arising from knowledge and experience rather than manual practice, and a considerable aptitude for organization (if the machine is to be used to its full capacity). The daily output and rate of expenditure are high. The actual laying techniques themselves are also very different—so much so that it is often disadvantageous to employ men who are skilled hand layers upon machine laying work, because they frequently carry with them hand laying practices which are prejudicial to the machine-laying technique. Similarly, the supervisory capacity required is of a much higher order. Whereas the hand-laying gang could safely be left in the charge of a competent foreman, the mechanical finisher requires, and should have, much more highly qualified supervision if it is to be employed fully and to the best advantage. It is not uncommon for a finisher to lay some hundred of tons of surfacing materials in a day and it is essential that supervision appropriate to this rate of progress and expenditure should be provided. Furthermore, the technique of specifying and manufacturing surfacing materials has advanced today to a stage of almost complete reliability, so that the operation of laying, which can produce such a great difference in the quality, cost, and life of the finished surface, is thrown into prominent relief. Not only is the riding quality of the machine-laid surface much better than that of hand-laid work but there are certain fundamental differences between the surfaces produced, the laying technique, and the rate of progress, which are of much wider implication. Hand-laid surfaces contain many irregularities of short amplitude and length and the traffic impact stresses which are induced by such irregularities have a marked effect in aggravating the irregularities themselves and promoting the disintegration of the road surface. Machine-laid surfaces, on the other hand, by reason of the automatic levelling and smoothing principles on which most of the finishing devices are based, do not contain irregularities of this nature and the riding quality of the surface is improved and its life prolonged. Furthermore, whilst it was virtually essential in the days of hand laying to prepare the base to a profile corresponding to that of the finished surface (because of the unconscious tendency of the layers to follow the base contour, and of the inability to produce good or uniform compaction before the surface was rolled, so that, on compaction, differential consolidation occurred according to the thickness, and the finished surface approximated in irregularity to the base contour, though to a lesser degree) the mechanical finisher will compact varying thicknesses to uniform density, within normal limits of construction, and will strike-off the surface to a high degree of accuracy, and differential compaction does not occur under the roller. Indeed, it has been found that the effect of traffic is frequently to lessen the type of minor irregularity which may be produced by the mechanical finisher so that the riding quality of the surface tends to improve and not to deteriorate. The effect of this on the technique and cost of the base preparation is considerable, though it is perhaps necessary to emphasize that in the use of these techniques great care should be taken in the provision of adequate drainage for the base on which the surfacing material is laid, so that such techniques may not be brought into disrepute by the use of incompatible materials in the base and top surfaces or by the use of unsatisfactory methods.

The economic implications of the change in conditions produced by the use of the mechanical finisher are interesting and far-reaching. In the days of hand laying the basis of selection of surfacing material was compounded of suitability to the site, base and traffic conditions, together with convenience and cost. At a time when the rate of laying of a representative hand-laying gang was relatively small, convenience was not the least of these considerations and frequently led to the choice of a material which would not require to be renewed for a long time. Such materials are commonly expensive and the possibility of renewing such a surface because the riding quality had deteriorated, or of abandoning it in favour of a better alignment, was not lightly to be entertained. Now that surfacing materials can be laid very cheaply, and much more easily and quickly, the engineer is not confined in his outlook or choice by these restrictive considerations. It is clear that the advantages of laying considerable lengths of surfacing material quickly and cheaply, on lightly prepared bases, are eminently suitable to our present economic conditions, when almost every effort must be directed to the preservation of the existing road structure, though it is equally evident that the extreme facility of these operations may lead to an undesirable over-investment in the existing road system if the present financial stringency persists and these practices have to be followed for too long a time. This is not to say that the long-life surface is outmoded, for the factors of convenience and economics must always be taken into consideration according to circumstances.

All mechanical surfacing appliances are similar in so far as they provide for the discharge of surfacing material from lorries into hoppers on the machine and incorporate provision for spreading this material in a uniform ribbon, to a desired width and contour, at the other end as the machine moves forward. They may be differentiated, however, according to the method used for striking-off the surfacing material when it is discharged from the machine. There are two basically different methods of striking-off the surfacing material to the correct degree by a spreader or finisher. In the first of these the material is struck off in a loose condition, sometimes with a bar or screed oscillating in the horizontal plane. When surfacing material is being laid on a base which is exactly parallel to the desired finished surface, this method produces a freedom and facility of operation which frequently leads to large quantities of material being laid satisfactorily in the course of a day; but when the surfacing material is being laid, as is most commonly the case, on an irregular base not parallel to the desired finished surface the method suffers from the major disadvantage that varying thicknesses of material are being laid in an uncompacted state and that, after compaction (which will vary according to the thickness laid) the base irregularities will become apparent in the surface after it has been rolled, according to the variation in thickness laid. Levelling devices used for correcting longitudinal irregularities with loose-strike-off spreaders or finishers must also necessarily make use of some type of proportioning device, located between the point of strike off and the wheels or crawlers, for the correction of base irregularities. These are both serious disadvantages in any attempt to produce a finished riding surface of high quality. An appreciation of this fact gave rise to the development of mechanical spreaders using a compacted-strike-off principle and controlling longitudinal compensation by means of a screed riding on the compacted surface immediately behind the point at which the material is struck off and compacted. This screed is guided by long arms that feel-out changes in grade ahead of the point of strike-off without reflecting small base irregularities in the compacted surface.

Apart from the types of equipment using the loose-strike-off principle, which have already been described, there are three main types of compacted-strike-off spreaders, or finishers as they came to be known from the fact that so little work required to be done after them, in common use in the continents of Europe and America—the Adnun, Jaeger, and Barber-Greene.

The principle of the Adnun paving machine is that the regulation of a smooth riding surface is obtained by the balancing of the weight on a roller at the rear of the machine, which travels on the finished surface of the carriageway, and is wheel-driven and steered. The surfacing material is discharged into a hopper with a revolving screw which distributes it directly on to the road surface and adjustments are provided for correcting and maintaining course thickness. The discharged surfacing material is levelled off in front of a screed by an oscillating cutter bar which has a transverse motion and is electrically heated. No consolidation or compaction is, however, obtained during spreading which is required to be undertaken by independent rolling behind the machine. It is capable of producing even running surfaces and good results have been extensively obtained with it.

The Jaeger Bituminous Paver is a development of the original Jaeger Spreader Box which was first brought out about 1934. It operates on the loose-strike-off principle and the regularity of the surface of the material being laid is maintained by a screed fitted with a series of teeth, oscillating horizontally at 666 strokes per minute, mounted on a pair of 12-foot-long skids at the sides of the machine. These skids, or equalizing runners, ride on the subgrade or base and the compensating principle is controlled by levelling pans floating on the mat already laid by the centre screed and holding the rear screeds to the same level by actuating sensitive hydraulic rams.

The Barber-Greene Finisher, which is by far the most common machine in use, was originally introduced in America in 1937 after a period of 7 years' trial and development and is now manufactured in Great Britain under licence. Essentially, it consists of two main units, tractor and screed, connected by a long compensating arm, upon which the longitudinal levelling principle of the machine depends. The tractor unit provides the motive power through crawlers travelling on the road base and includes the receiving hopper, spreader screws, power plant, transmissions, guide and feed controls, and operator's seat. The screed unit is towed by the tractor unit through lever arms which extend forward from the screed unit and pivot on the crawler of the tractor unit. The tamper, thickness controls, crown control, screed heater, and the screed itself are all part of the screed unit which rides on the finished compacted surface. The mixed materials are dumped into the receiving hopper from a truck which is slowly pushed ahead by the finisher bumper rolls as paving continues, without any interruption in the course of supply being necessary. At the bottom of the hopper are two independently operated bar feeders, which convey material from the hopper to the spreading screws and also control the amount of material ahead of the tamper. The spreading screws uniformly distribute the mixed bituminous material across the full width of the mat, the tamper compacts and strikes off the material, and the screed, following immediately behind, completes the smoothing or ironing of the surface. The vertical travel of the tamper is $\frac{1}{8}$ inch at 1,200 impact strokes per minute, and when the tamper is in its lowest position it projects slightly below the bottom of the screed. This fixes the surface elevation just before the screed rides on to it; thus there is no variation in material density as the tamper is the actual point of strike-off and the screed merely irons the compacted surface. The actual tamping, or consolidation, is done by the front inclined surface of the tamper as it descends, compacting the material to a uniform density and allowing the excess to push forward and upward. A continuous supply of material in front of the tamper ensures uniform compaction regardless of variations in the subgrade or of the actual mat thickness, within normal limits of construction.

To understand the levelling and compensating action of the Barber-Greene Finisher it must be appreciated that when the whole machine is on an even base the tractor unit pulls the screed unit with the bottom of the screed parallel to the direction of pull exerted by the crawlers. When it is desired to increase the thickness the screed is tilted up by the thickness controls and gradually rises to a higher level until it again becomes parallel to the direction of pull. Conversely, to decrease the thickness, exactly the opposite takes place; sudden changes in grade are therefore impossible and the surface remains smooth. This compensating action is brought into play automatically when the machine encounters a change in grade or, what is really equivalent, an irregularity in the shape of the base on which the machine is travelling. On encountering an upgrade the change in crawler elevation tilts the screed so that the mat becomes thicker through the arc of the upward turn until the screed again assumes a position parallel to the direction of pull at the prescribed mat thickness. The reverse action takes place when the machine encounters a down grade. In both cases the compensating action takes time and distance to come into effect so that the transition is gradual and a smooth surface is obtained. The standard laying width of the finisher is 10 feet, though the width of the mat can be extended to as much as 12 feet or reduced to 8 feet by the use of extensions or cut-offs fastened to the screed unit. Eight feet is the minimum width which it is desirable to lay with the finisher though it is possible in an emergency to lay a narrower width by tightening up one side of the screed support spring or by supporting one side of the screed on a board, shoe, spring, or previously laid strip and to work with only one feeder. In such circumstances screed levelling control is difficult and the practice is not generally recommended. A smaller version of this machine, similar in basic principles, design, and function, has recently been introduced and lays a standard overall width of 7 feet as compared with 10 feet for the standard machine. This machine will lay a surfacing mat having a width ranging from 5 feet 6 inches to 8 feet, adjustment being made by means of extension plates or cut-off shoes, and has been introduced primarily for use on narrow lanes which the standard machine could not negotiate.

The variation in surfacing width, on both machines, can be made in steps of 3 inches and most widths of road can be laid in multiples of these dimensions. Where this is not possible, or where the road varies in width, the machine should be kept on a straight course and the narrow widths at the side filled in by hand or by bleeding out from the side of the machine, with the side plate, or plates, removed.

Economics

Very great differences in riding quality, output, and cost can be produced by efficient use of the finisher and proper organization of the work, and it is a point for consideration as to whether present British methods of mechanical surfacing of roads represent little more than an improvement in the riding quality of the surface without reflecting the capacity, or potential capacity of the machine in reduced costs. For example, in Great Britain a daily output of the order of 100 tons laid is commonly regarded as adequate, 200 tons as satisfactory, and 300 tons a day as exceptional. These figures should be compared with the output accomplished or achieved by the same machine in the United States, usually operated by a far smaller gang, where 700 or 800 tons a day per machine is a normal tonnage and up to 1,200 tons per machine per 8-hour day is not uncommon. An output of that order is not to be considered as fantastic or outside the realms of possibility, as may be seen from a brief consideration of the quantities that can theoretically be laid by a finisher in an 8-hour day. If it be assumed that the machine is laying a mat 12 feet wide composed of material which can be laid satisfactorily in third gear, high operating speed, that is, at a speed of 30–36 feet per minute, then the daily output per inch of thickness is about 1,100 tons, that is to say 90 tons per foot width per inch of thickness in an 8-hour day. This would represent the rate of laying for carpet coat work, whereas the daily rate of laying for a base or a thicker surface might vary between 2,200 and 3,300 tons per 8-hour day for a range of thicknesses of between 2 and 3 inches. If this, then, is the theoretical capacity, an average output of 1,100 tons per day should be reasonably achieved, so that 200 tons per day represents a very low relative standard of efficiency. The difference between good and bad production figures depends primarily upon efficient organization and use of equipment. Production studies made by the American Bureau of Public Roads on bituminous finishers working on active surfacing jobs on rural highways have revealed that, even under American conditions of operation, delays might amount to as much as 79 per cent of the total available working time. A primary analysis of the total time spent on the work showed the distribution given in Table 2.

TABLE 2.—DISTRIBUTION OF 1,480 HOURS TOTAL AVAILABLE WORKING TIME FOR BITUMINOUS FINISHERS ON EIGHT ACTIVE BITUMINOUS SURFACING PROJECTS ON RURAL HIGHWAYS

Time element	Percentage of total available working time	
	Range	Average
Total available working time	100	100
Major delays	28–77	57
Net available working time	23–72	43
Minor delays	12–43	22
Actual productive time . .	11–33	21

Major delays may be further broken down into the causes detailed in Table 3.

The classification of minor delays is shown in Table 4. Although minor delays are, in general, only a few seconds each in duration, it will be noted in Table 2 that they amount to 22 per cent of the total available working time. The full extent of minor delays can usually be better visualized by comparing them to the net available working time. For example, when minor delays are expressed as a percentage of the total available working time, a job having frequent and extensive major delays, such as

TABLE 3.—CLASSIFICATION OF MAJOR DELAYS TO BITUMINOUS FINISHERS

Nature of Major Delay	Percentage of total available working time
Weather: rain, wet, cold	23
Plant stopped owing to shortage of materials	12
Delays due to auxiliary operations: preparation of sub-grade, hand finishing of turnouts, etc.	9
Lack of bituminous mix at finisher	4
Plant repairs and adjustments	3
Finisher maintenance and repair	3
Moves on the project	2
Other	1
TOTAL	57

TABLE 4.—CLASSIFICATION OF MINOR DELAYS TO BITUMINOUS FINISHERS

Nature of Minor Delay	Percentage of net available working time
Lack of bituminous mix at finisher	43
Delays due to auxiliary operations	4
Finisher, maintenance, repair, and adjustment	2
Personnel	1
Short moves	1
TOTAL	51

bad weather, tends to show a lesser percentage of minor delays than a job having a few major delays. Thus, a better basis for indicating the extent of minor delays in relation to the production operation is to express them as a percentage of the net available working time. For the finishers employed on the eight jobs which were studied, the minor delays varied from 32 to 60 per cent of the net available working time, with an average of 51 per cent. Lack of bituminous mix accounted for the majority of the minor delays to the finisher.

Other minor delays, arising from interference by traffic and so on, were negligible on the jobs studied.

An attempt was made to classify both major and minor delays in accordance with their basic cause, but in those cases where the basic cause could not be determined, the delays were classified in accordance with the apparent cause. For example, the basic cause of a delay to the finisher resulting from a lack of bituminous mix may have been caused by an actual shortage of hauling units or perhaps by another delay elsewhere on the job; but if the cause of this delay could not be readily determined by the observer, the delay was charged to the apparent cause—lack of bituminous mix at the finisher.

The importance of efficient organization of transport serving the machine is shown from the fact that lack of bituminous material at the finisher accounted for not less than 43 per cent of the minor delays. As an illustration of the importance of transport in the cost of surfacing materials it may be said that the total cost of surfacing materials, laid on the road, is divisible into the following elements :—

(1) The cost of the raw materials at the plant.
(2) Plant charges :
    (*a*) Production.
    (*b*) Laying.
(3) Operating costs—fuel and labour.
(4) Transport.

On any day's work item : (1) is a constant charge per ton, fixed under the supply tenders ; items (2) and (3) are variable, being inverse functions of the daily output ; item (4) is a variable dependent in a major degree on the efficiency of the organization. The cost per ton is :

$$\text{cost of material} + \frac{\text{standing charges and operating costs}}{\text{daily output}} + \text{transport.}$$

Consider, for example, the costs at a typical plant producing surfacing materials, originally rated to manufacture 80 tons of dense asphalt per day. The plant can, and usually does, produce materials which are less difficult to manufacture at outputs greatly in excess of this figure. The nominal output of 80 tons per day stands at about the 45 degree tangent to the output/cost curve ; greater outputs decrease in cost per ton at a diminishing rate, whilst smaller outputs increase in cost at an increasing rate. This point is of great significance in arranging the organization of the plant, since it is more important to achieve a constant output than to have large fluctuations resulting in the same average output. The reduction in cost which it is possible to obtain by increasing the output diminishes very rapidly as the output increases beyond 150 tons a day, and for increases in output beyond 250 tons a day the reduction in cost per 10 tons increase in output is comparatively small, being 6*d*. per ton for an increase from 250 to 260 and 4*d*. per ton for an increase from 300 to 310 tons per day. It is difficult to increase these terminal outputs and since, with an output of 300 tons a day, each lorry surplus to requirements represents an unnecessary increase in cost of 3*d*. a ton, the value of a careful study of transport organization is very obvious. The behaviour of road transport serving a bituminous finisher can be analysed or predicted by means of a diagram such as that in *Fig. 1*, which shows the process of supply to two comparable schemes. It will be seen that the left hand ordinate of the diagram is divided into three sections, composed of the plant and each of two jobs which were being served by the plant at the same time. Each of these jobs consisted of shaping the existing road in 1½-inch single-course material,

Fig. 1

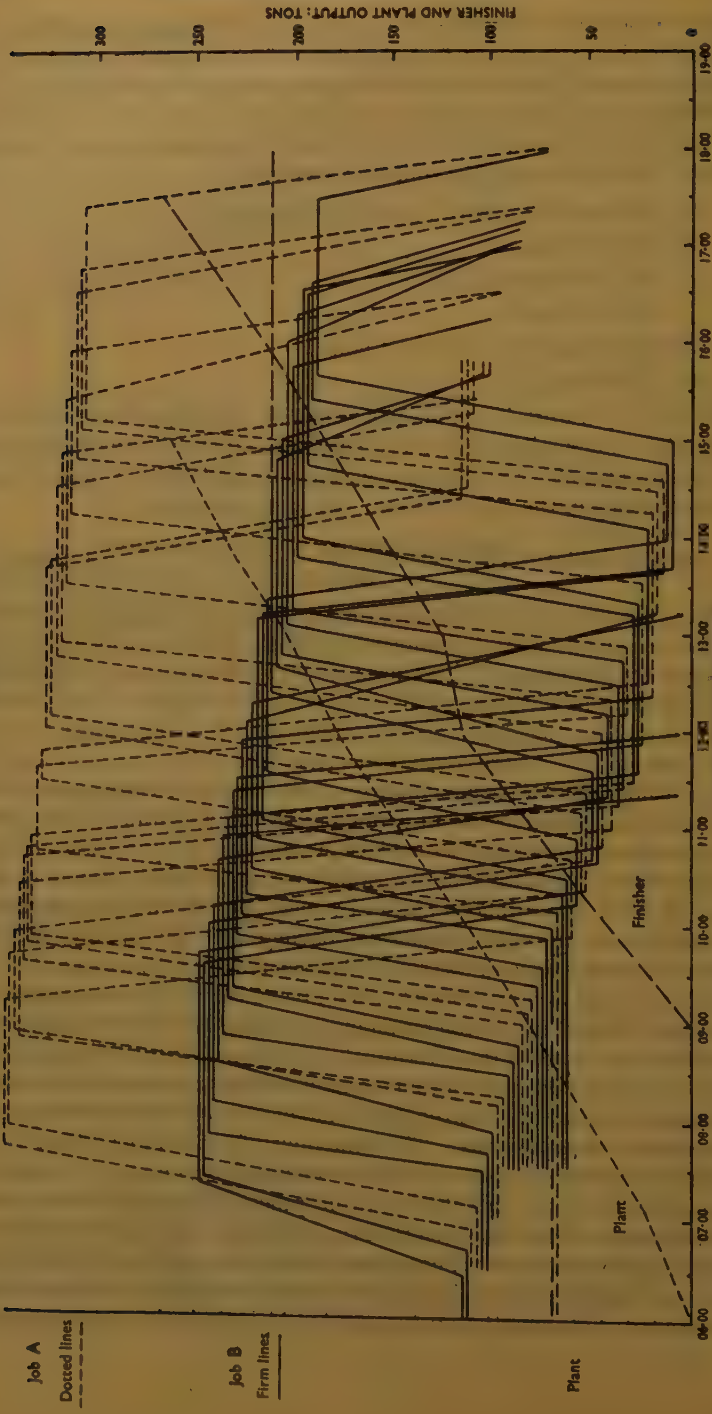


DIAGRAM SHOWING TRANSPORT MOVEMENTS

and they were both approximately equidistant from the plant, so that, from the point of view of organization, they may be fairly compared one with the other. The base-line of the diagram is divided into the hours of the working day. Each of the lines on the diagram represents the movement of one lorry. Horizontal lines represent the periods during which the lorry is standing at the plant waiting to be filled, or on the road waiting to be unloaded. To ensure continuous operation of the plant and the Barber-Greene it is normally necessary to have two or three lorries overlapping so that if one of them should fall out or be delayed there will be no interruption of production at the plant or of the work of the finisher.

The following points emerge from the diagram :

(1) The sloping " travelling " lines represent the period taken by each lorry in going from the plant to the job and vice versa. Since they are all parallel it is apparent that transport behaviour on the road has been good.

(2) A vertical intercept at any point on the diagram will show the number of lorries standing, and also the number of lorries which has to be in the supply line itself in order to keep the organization going. It will be seen that job A was fairly well organized and that at no time was there an excessive accumulation of lorries on the road, although there was a short break of about a quarter of an hour at about 11.45 a.m., when the road gang ran out of material and had to wait for the arrival of another lorry. Job B, on the other hand, was evidently over supplied with transport. At times, there was as many as eight lorries on the job site, though an accumulation of only three was all that was necessary. A great deal of transport time is wasted between 7.30 and 10 o'clock, during which time a large number of lorries are waiting to take their first load. This is sometimes unavoidable since it is difficult to hire lorries for a shorter period than a complete day.

(3) The right-hand ordinate is related to the diagonal lines emanating from the left-hand corner of the diagram, which show the cumulative output from the plant and the cumulative tonnage laid by the Barber-Greene, hour by hour. From an inspection of these lines it is possible to deduce whether any unforeseen circumstances, such as a breakdown at the plant or of the finisher on the road, has been responsible for an unusual accumulation of lorries at any time.

Studies made in the United States on transport hauling bituminous mixes from the plant to the finisher on bituminous surfacing projects show that the total time each truck spends at the plant, at the finisher, and in miscellaneous waits and delays averages more than ½ hour for each round trip. The average road speed was observed to be about 30 miles per hour.

The following information was obtained from studies made by the Production Cost Unit of the United States Bureau of Public Roads on nine different projects in the eastern and south-eastern States over a period of 20 months. About ninety-five trucks of various sizes and makes were observed for a total of 1,050 truck hours of operation. The average load carried by the trucks was 7·4 tons, and the average haul distance was 7·9 miles. Most of the hauling was done over surfaced roads that were well maintained.

Table 5 is a summary of data relating to the major elements of the truck cycle.

TABLE 5.—SUMMARY OF CYCLE DATA RELATING TO THE OPERATION OF TRUCKS HAULING BITUMINOUS MIX ON BITUMINOUS SURFACING PROJECTS.

Element	Range	Average
1. Time in plant per trip, excluding waits and delays	3·8–16·1 minutes	8·2 minutes
2. Time at finisher per trip, excluding waits and delays	3·0–7·9 minutes	5·3 minutes
3. Waits and delays per trip . . .	10·6–26·7 minutes	18·6 minutes
4. Total time constant (sum of items 1, 2 and 3)	22·0–50·3 minutes.	32·1 minutes
5. Haul speed, loaded	16·3–37·6 m.p.h.	27·8 m.p.h.
6. Return speed, empty	22·3–41·8 m.p.h.	32·3 m.p.h.

TABLE 6.—OPERATING CYCLE ELEMENTS AT THE PLANT AND AT THE FINISHER FOR TRUCKS HAULING BITUMINOUS MIX ON BITUMINOUS SURFACING PROJECTS.

Cycle element	Average time: minutes
1. Time in plant, per trip	
a. Loading time	6·3
b. Manoeuvring, weighing, covering the load, etc.	1·9
Total time in plant, exclusive of waits and delays . . .	8·2
2. Time at finisher, per trip	
a. Discharge load	3·8
b. Manoeuvring, exchanging, etc.	1·5
Total time at finisher, exclusive of waits and delays . . .	5·3
3. Waits and delays, per trip	
a. At the plant	14·8
b. All other	3·8
Total waits and delays	18·6

Delays to the trucks during periods when the plant or the finisher is shut down because of repair, material shortages, moves, and so on, have been excluded when the shutdown is 15 minutes or more in duration.

The ranges shown in Table 5 are job averages. On several of the projects the variations from day to day frequently exceeded the ranges which are shown. For example, the haul speed on a particular job may have varied from 12 to 20 miles per hour, but only the over-all average for this job was considered when listing the entries in Table 5.

For the jobs which were studied, it was observed that, in general, the faster haul and return speeds were obtained on the jobs having the longer haul distances.

A partial breakdown of the times involved in various operations at the plant and at the finisher is shown in Table 6.

The above items are largely self-explanatory. It is interesting to note, however, that the average time required to obtain a load at the plant was 6·3 minutes, or 66 per cent greater than the time required (3·8 minutes) to discharge the load at the finisher, and that the rate of loading was approximately a ton a minute, the rate of discharge at the finisher being about double this figure. A comparison between British and American rates of work is illuminating. On the production side, the British surfacing materials plant already referred to, though originally designed for an output of 10 tons per hour of dense asphalt with a 1-ton mix will, under pressure, produce 25 tons per hour of coated stone. It takes eleven to thirteen men to work it. An American plant of a similar size and age will produce 160 tons an hour, through a 2½-ton mixer and is run by five men. The tonnages produced per man-day are: British 18, American 256. Though comparisons between rates of pay in the two countries must be qualified by many considerations it is interesting to note that, at present rates of exchange, the American operator is paid six times as much as his British counterpart but produces fourteen times as much.

In short, there seems to be room for improvement in British methods of organization and use of equipment, and little increase in output or reduction in costs is to be expected until these matters have been attended to. If a general comment may be hazarded, it seems, unfortunately, to be part of British outlook and tradition to spend much time and labour on refinements and minor improvements in quality to the neglect of production and cost studies which, in their effects, would far outweigh the economic benefit to be derived from academic research. In matters such as the development of surfacing mixtures suitable for fast laying, the practising British engineer is hampered by the lack of qualitative tests which would enable him to express the desired characteristics of the laid surfacing materials and to relate them to economic values, and he is therefore forced to rely on tradition and experience. Such tests have been developed in the United States and American engineers are confidently going ahead on that foundation. Whether they are wholly justified in doing so, only time can show, but there is as yet no reason to believe the contrary. Some work alon these lines is now being done in Great Britain. Until fundament research can provide adequate guidance it is to be hoped that researc on the final properties of surfacing material will be treated as being primary urgency.

Base Preparation

Turning now to the actual surfacing operation, it may be said tha where the base is not parallel to the desired finished surface proper preparation of it is fundamental to the laying process. The base should b examined some time before laying begins, to note and mark surfac defects so that faults which cannot be corrected by a single passage of finisher may be rectified in advance. (This and the remarks which follow concerning base preparation refer only to the shape of the base and no to its strength, which is assumed to be adequate.)

In order to avoid delay and to produce a satisfactory surface it is essential to plan the method by which each irregularity is to be corrected whether it be by preparation work or by adjustment of the controls whil laying is in progress. For this purpose irregularities in the base may b divided into three classes :

(1) Minor irregularities and depressions which will be corrected by the automatic action of the machine.

(2) Long but not deep depressions which, if not corrected by manua adjustment of the thickness controls, would be only partially smoothed out by the automatic action of the machine.

(3) Deep irregularities long enough for the crawlers to drop into and run along for a distance before climbing out again ; preparation work will be required for the correction of irregularities of this nature.

A base which contains only minor irregularities, and has no longitudinal or transverse depressions which cannot be spanned by the machine's crawlers without a change of grade, does not require manual adjustment of the thickness controls to rectify the irregularities. The correct thickness of mat should be set at the start and if the base is otherwise true no alteration of the controls should be needed. This type of base is rarely encountered.

Major irregularities of the second type which are not deep but cannot be spanned by the machine's crawlers, can sometimes be corrected by manual operation of the thickness controls without previous preparation. If the controls are not altered on a base containing irregularities of this type the finished surface is smooth but has long undulations and may cause sidesway on a vehicle having more than one track because of the crawlers changing level differentially. To eliminate sidesway, which can be a serious fault in a machine-finished surface, and to prevent undulations, the depressions between high points of the same level or plane should be filled in beforehand. When shaping such a base by machine the method normally used is to measure, by means of levels, templets, straight-edges, or boning rods, the depth of material needed at given points, at intervals of not more than 10 yards, and to mark these thicknesses on the road so that they can be seen by the operator. From these readings the thickness controls can be manually adjusted to lay the correct depth of material at the given points.

Any adjustment of the thickness controls should be gradual and made in advance. The machine takes a certain time to act, approximately 10 to 15 feet in travelling distance (it is on this feature that its levelling or smoothing action is based), and unless this delay is anticipated the required thickness of mat will not be obtained at the desired spot. If the controls are moved too much or too often before this time has elapsed the result is over-adjustment and an undulating finished surface. The standard of finish, in this case, depends mainly on the ability of the operator; inexperienced operators, by poor judgement and improper adjustment of the thickness controls, can produce more undulations than if the machine were permitted to lay the surface with fixed controls.

When dealing with deep irregularities of type (3) the above method cannot be applied with any degree of success. Where the holes are deep and long enough for the crawlers completely to drop into and run along for a distance before climbing out again, the depression cannot be rectified by manual adjustment of the thickness controls. In this case it is essential to provide the finisher with a regulated surface to run on, either by levelling up the base with added material or possibly by filling in enough of the depression to give the crawlers a track to run on in order to maintain the machine at its correct level.

By careful reconnaissance and diligent application of each of these methods, as required, a surface which is transversely and longitudinally true can be obtained.

The thickness of the mat which is laid naturally varies on irregular bases, being thicker in the deep spots and thinner on the high points. It is therefore incorrect to adjust the thickness controls if the mat appears thicker in one place than another. To maintain minimum thickness and a level surface should be the aim. The material used should be measured and paid for by the ton, so that there will be no inducement to depart from this principle. The temperature of the mixture being laid may influence the thickness. If there is a wide temperature variation between two loads the stability of the mixes is different and causes poor levelling control. It is claimed that compaction by finisher is effective for a depth of up to 6 inches, but a good guide to the thickness that can be laid in one application may be obtained by limiting the maximum thickness of each layer to 1½ times the normal thickness of the course. This takes into account the nature of the material being laid, since the grading and composition will be related to the normal thickness, and the principle has been foun to work well in practice. Superelevation can also be applied, in successiv increments if required, by the same method.

When laying surfacing materials by machine, side battens to retai or regulate the height of the material being laid are *not* required. Indee if they are used they will interfere with the action of the finisher and pr vent the automatic compensation on which its action depends.

Operation of the Finisher

Before laying begins, the spreading machine should be cleaned, prepared for the width of the mat to be laid, with proper extensions, etc and carefully inspected for mechanical defects. The feeder mechanis should be examined to ensure that the correct volume of material will b distributed evenly across the width required. Tamping and/or screedin devices should be checked for alignment and cleanliness. When diese oil is used for cleaning the machine, care should be taken to ensure tha the oil does not come in contact with or damage the surfacing material If the base contains a bituminous mixture the machine should be move off the road before cleaning is begun.

The screed should be heated before laying is started, to prevent materia from sticking to the screed plate and causing a drag or tear. A perio of 20 or 30 minutes is normally sufficient to warm the plate to a prope temperature, but this is dependent on the efficiency of the burner and th weather conditions. During laying, heat should be applied to the scree as and when required by the type, condition, and temperature of the mixture being laid; care should, at the same time, be taken not to overhea the mixture, or burning and roughening of the mat will ensue.

The spreading machine should then be moved into position for laying and the controls set for the thickness and profile required for each strip. This should normally be carried out by lowering the screed unit on to blocks of approximately 20 per cent greater thickness than the compacted depth desired, set to correct profile. The thickness control settings should then be turned to the correct position for maintaining this thickness. If a start is to be made from previously laid material, the setting blocks are not necessary and the screed should be lowered on to the joint so that the leading edge is 3 or 4 inches to the rear of the joint edge, the thickness-control settings being adjusted to allow for any additional compaction on rolling. A guide line should be indicated to the operator, along which the machine should travel. This line should be on, or parallel to, the centre of the road. The moving screed outlines an inclined plane in space and this must necessarily be longitudinally true if a correct profile is to be obtained. Any deviation from this guide line should be gradually corrected by a light movement of the controls. Excessive turning of the machine will mark the finished surface and spoil the profile. Slight correcting movements are all that should be permitted.

The supply of material to the machine should be so arranged throughout the day that there is the minimum number of stoppages, because stopping and re-starting the machine is a frequent cause of surface irregularities and defects. Certain mixtures will not support the weight of the screed unit and a prolonged halt will cause the heavy screed to indent and mark the finished mat unless the screed is raised. Further, material should not be held too long in the machine, because hot mixes cool and partially set if allowed to remain long in contact with the metal interior. This invariably results in dragging of the partially set material and holes appear in the finished surface. Skilful operation can minimize the effects of these irregularities but, by ensuring continuity of the supply of the material, they can be avoided.

In normal circumstances, a spreading machine is capable of laying material faster than it can usually be provided and, in order to have a continuous supply while the finisher is in operation, there should be an accumulation prior to the commencement of laying. This accumulation of material should preferably be the minimum amount necessary to ensure continuity throughout the laying period.

When laying steep gradients it is better to operate uphill in order to prevent the spilling of material from the machine hopper, and to assist the control of machine and vehicles.

It is absolutely essential that the material arriving at the laying site should conform to specification and be consistent both in composition and temperature. Any variations in these respects will affect the finished product. The conservation of heat in a bituminous or asphaltic mixture when in transit from the plant to the laying site is a matter of prime importance. It ensures that the mixture can be manufactured and laid at its optimum temperature, without overheating of the constituent materials at the plant in order to allow for any temperature drop which may take place in transit, and preserves the whole of the mass at a uniform temperature so that it exhibits uniform characteristics as between load and load, and the parts of a load, when being laid. Mechanical finishers are sensitive to temperature variations in the material being laid and require adjustments in travelling speed and screed heat to compensate for them. Such adjustments cannot be made instantaneously, and sometimes not without halting the finisher, and temperature fluctuations and adjustments of this nature invariably result in blemishes and imperfections in the finished surface. The finisher cannot, under any circumstances, successfully lay material of which parts of the same hopper-load are at markedly different temperatures. The extent and effect of the potential temperature-drop in transit depends, obviously, on the difference in temperature between the atmosphere and the mixture itself, but it is also dependent on the typical cooling curve of the mixture and its ductility/temperature characteristics, its degree of contact with the sides of its container, and the insulating properties of the material of which the container is made.

Wooden-sided lorries conserve the heat better but sometimes make it difficult to discharge the material owing to their higher co-efficient of friction. Wooden-sided steel-lined lorries fulfil both requirements but increase the unladen weight and diminish the pay-load. Experiments are now being made with light alloy bodies of hollow section in an effort to combine strength, low frictional resistance, and good thermal insulation. In low temperatures, the top of the load should not only be sheeted as a protection against the weather but should be double-sheeted to protect the material against loss of heat from this source. Blankets consisting of kapok quilted between tarpaulins have been used with great advantage.

It should be noted here that specifications suitable for materials to be laid by hand may require slight variation in the binder content and penetration for machine laying. The vibratory compaction provided by the machine and the fact that the material is sometimes laid at a higher temperature together result in a thinner film of binder being left on the aggregate. Any excess binder, such as might result from an unadapted hand-laying specification, is brought to the surface and may result in a skin of binder which reduces the frictional resistance of the road surface. Slight reductions in binder content are usually sufficient to compensate for this difference in laying conditions, but even so, the material still remains a derivation from hand-laying days, and is not to be considered a mixture designed for machine-laying conditions.

On arrival at the site, the material should be in such a condition that it can be satisfactorily laid within a reasonable period of its arrival. All loads of hot or warm-laid material should be carefully sheeted to prevent loss of temperature in transit or during any waiting period. For long hauls, and in poor weather conditions, it may be necessary to use double sheets or kapok mattresses to keep the material in a fit state for laying. Metal-lined wooden-bodied vehicles should preferably be used for both high- and low-temperature mixes. A permissible temperature range at the laying site should be stipulated and any material which does not satisfy these conditions should be rejected. In practice, it is advisable to raise the mixing temperature of the first loads on the day. The reason for this is two-fold. Firstly, it allows for the loss in temperature caused by the wait while material is accumulating before laying begins and, secondly, the air temperature during the early hours of the morning is generally lower than the average throughout the day. These factors may not always be applicable but they should be borne in mind.

Care should be taken when discharging from the lorry to the hopper to ensure that no surface irregularities are caused. If vehicles are reversed against the machine and it is momentarily halted or pushed back, the rear edge of the screed unit is forced into the finished surface. This action forms a ridge across the mat. To avoid this, it is advisable to halt each vehicle slightly in front of the machine and permit the latter to advance and push the vehicle along. Vehicles should be reversed so that they are in the direct line of travel of the machine. Unless this is done, the vehicle tends to "steer" the machine off its intended course and necessitates turning to regain its original line. This often produces irregularities in the finished surface, such as low spots along a matched joint owing to insufficient overlap. The loads should be tipped into the machine hopper in such a manner that spilling of the mixture does not occur. If spilling is unavoidable, then the spilled material should be cleared away in front of the tracks or poor levelling will result. The machine treats spilled material as sudden changes in grade, and thus produces irregularities in the finished surface. The vehicle brake should be lightly applied so as to enable the machine to push it along without effort, but enough to prevent it from rolling forward, and, after discharging, all vehicle bodies should be lowered before the vehicle is driven away from the machine. The practice of driving off with the body raised may cause irregularities unless the interior is absolutely free from material, because small quantities of the mixture may sometimes adhere to the corners and sides of the bodies and may fall in front of the crawlers and upset levelling as previously explained. Irregularities can also be formed in surfaces that have already been laid, if such small quantities of material fall from the moving vehicle and are rolled in by the action of the traffic. When laying a thick mat the empty vehicle should be removed as quickly as possible to avoid delay in delivery of the following load, and to prevent the machine from stopping.

To facilitate discharge of the material it is permissible to oil or dust the interior of the vehicle but the amount of oil or dust used must be kept to the minimum. A combination of heavy fuel oil and a light sprinkling of coarse sand is commonly used when transporting dense mixtures such as hot-process-rolled asphalt. For tar and bitumen macadams a light dusting of quarry fines or precoated grit is usually sufficient but precautions should be taken to prevent the use of excessive quantities of these materials—patches of dry material may otherwise appear in the final surface and develop into potholes under the action of traffic; adhesion between the surfacing material and the base may also be impaired.

The machine should be operated at the speed most suited to the type of mixture and the rate of supply. If the maximum mixing-plant output is insufficient to keep a machine operating at peak-laying capacity, then it is advisable to select a lower operating speed than might otherwise be used. By doing so, continuous laying is maintained instead of a high but intermittent laying rate. An even flow of material should be fed from the hopper across the width of the machine, thus enabling the tamping and/or screeding mechanism to maintain a constant level throughout the laying period. Failure to feed sufficient mixture causes the surface to be thin and to tear; over-feeding results in excessive strain and wear on the machine. At intervals, dependent upon the type and condition of the mixture, the hopper should be almost cleared to prevent cooling of the material on the hopper sides. Low temperature mixtures with binders of cutback bitumen or tar do not require clearing from the hopper as frequently as mixtures which contain straight-run bitumens or fluxed lake asphalt. The latter type of mixture should be cleared frequently or the material will partially set, forming lumps which are not easily broken up by the machine and which eventually cause dragging or holes in the finished surface.

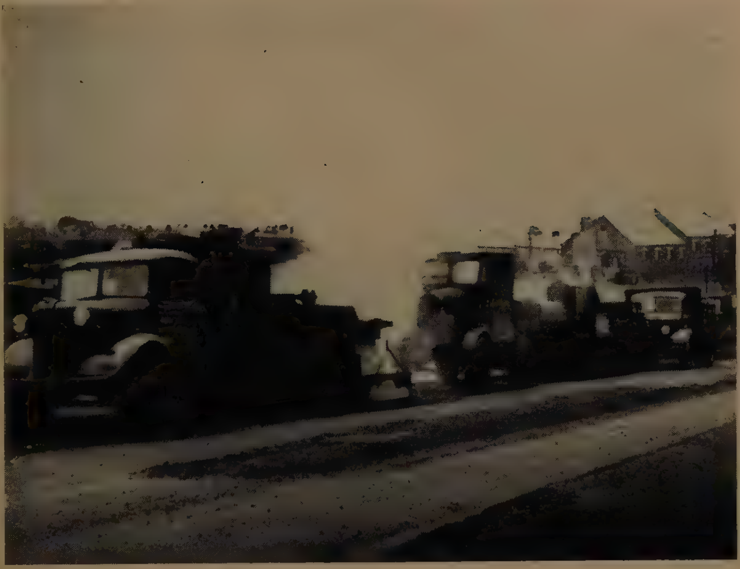
With tough mixtures it is sometimes advisable to remove the end plates from the machine spreader box; this prevents partially set material from packing tight against the end plates and interrupting the free flow of loose material to the screed. With the plates removed, any partially set pieces of mixture are gradually propelled to the sides, where they can be extracted by hand. These pieces would otherwise be dragged along until they broke up, and passing under the screed, would form holes in the finished surface.

Tough asphalt mixtures sometimes require the use of an agitator on the machine in order to ensure a satisfactory flow of the mixture to the screed. Then, when the machine is being operated, it is necessary to maintain, with the Barber-Greene Finisher, a clearance of 3 inches between the kerb-line and the agitator drive, which itself overhangs the left hand side of the machine by 7 inches. A total width of 10 inches will remain uncovered because the end plate on the agitator drive side cannot be raised sufficiently to permit bleeding-out of the material, and this must be filled in by hand.

When the width of surface to be laid is such that it requires more than one strip, especial care is necessary in matching adjacent longitudinal strips in order that the joint may be correctly formed and not become a line of weakness in the road surface. The technique to be used in making longitudinal joints is governed by the type of material being laid. When a plastic slow-setting material—either open or medium-textured—is in use, the best procedure is to roll the first strip within 6–12 inches of the longitudinal joint. The second strip should then be laid to the same depth as the unrolled part of the first. The rolling of the second strip is then begun at the free edge and continued progressively (rolling in a longitudinal direction) until the roller wheel is able to straddle at one pass the final section of the second strip and the unrolled section of the first. By this means, the second strip is pressed well home into the first by the slight lateral displacement which occurs on rolling and the joint of the first and second strips is properly fused. When tar or cut-back binders are used it is possible, in normal weather conditions, to lay considerable lengths of the first strip before matching the longitudinal joint and still obtain satisfactory fusion.

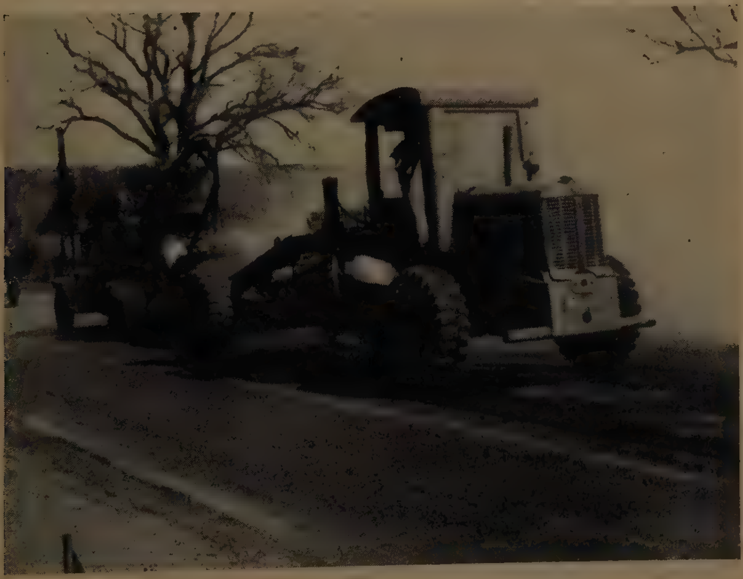
With material which is dense or hardens more rapidly, this technique cannot be followed because the first strip will have become too hard to allow fusion of the joint by the time the second strip is laid. It is therefore necessary to roll the full width of the first strip and lay the second strip thick enough to allow for compaction by the roller to the level of the first

Fig. 3



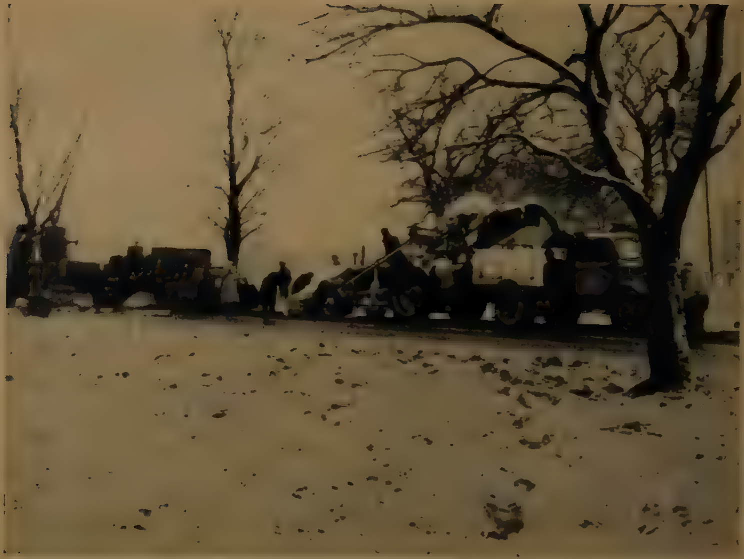
Heating and Planing Machines as Operated by Messrs Non-Impact Surfaces Ltd.

Fig. 4



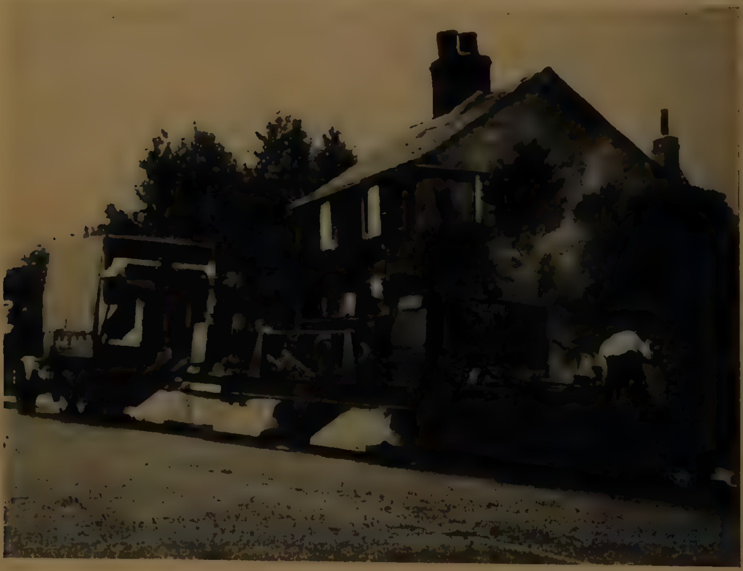
Power-Operated Blade Grader and Heating Machines as Operated by Anglo-American Asphalt Co.

Fig. 5



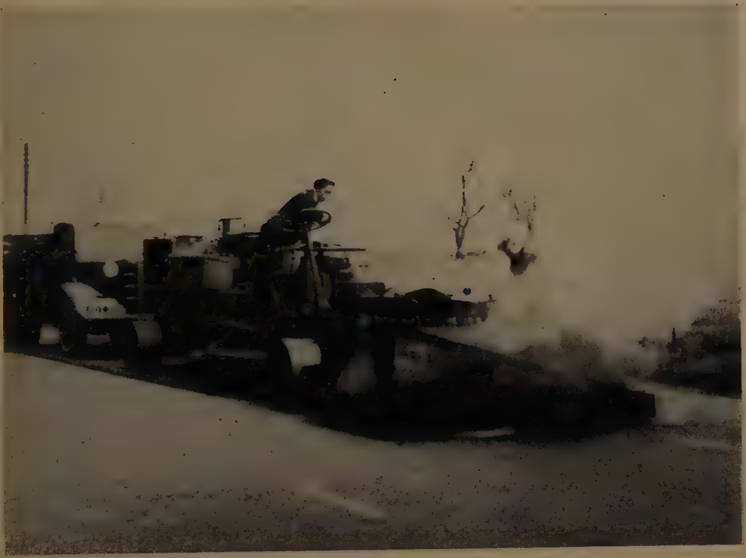
Heating and Planing Machines with Self-Loading Device as Operated by East Anglian Roadstone Corporation

Fig. 6



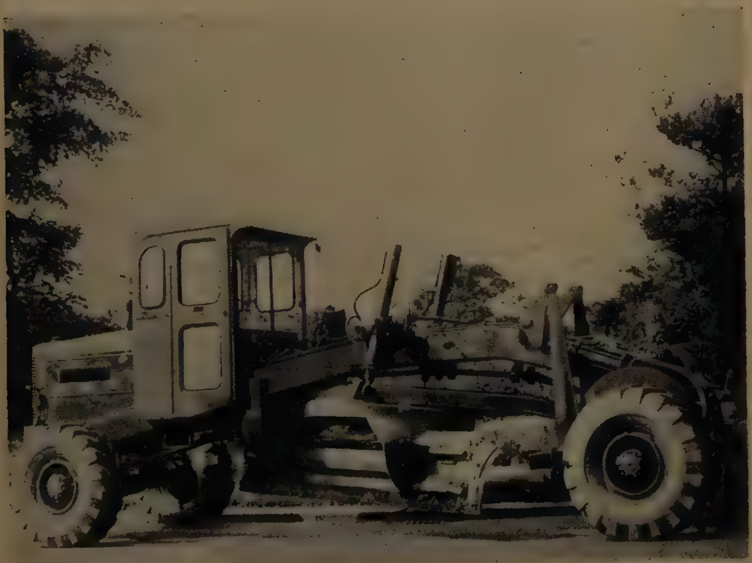
Heating and Planing Machine as Operated by East Anglian Roadstone Corporation

Fig. 7



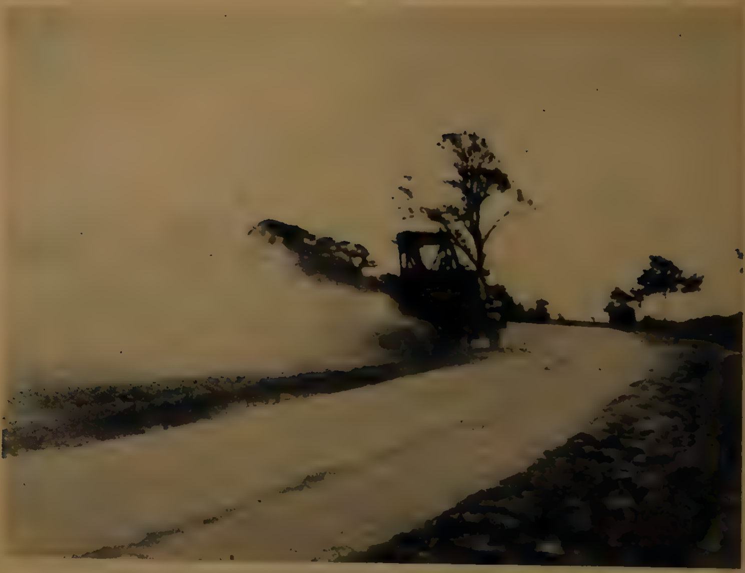
Heating and Planing Machine as Operated by Ameys Asphalt Ltd.

Fig. 8



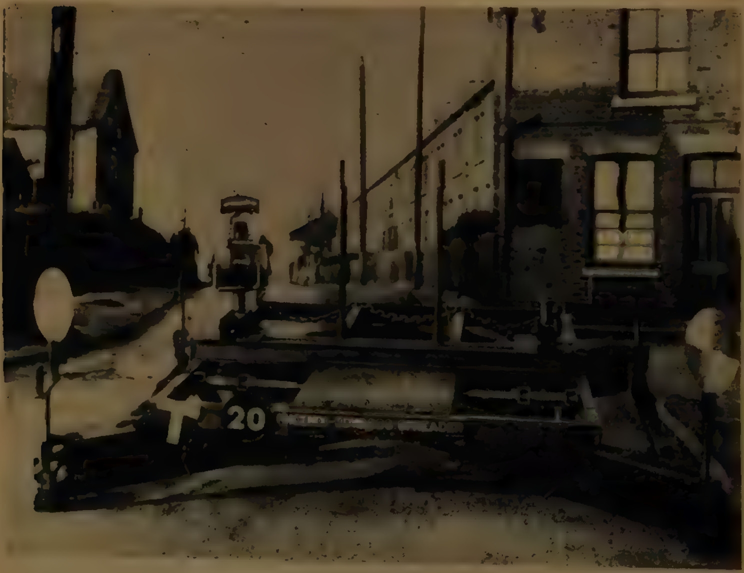
One of the Latest Type of Power-Operated Blade Graders now Manufactured in England

Fig. 9



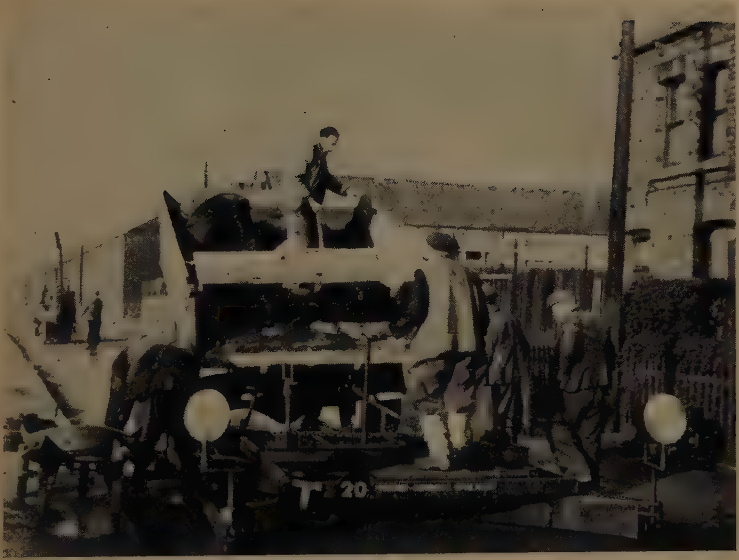
The Smoke Nuisance which might arise from Heating Operations

Fig. 10



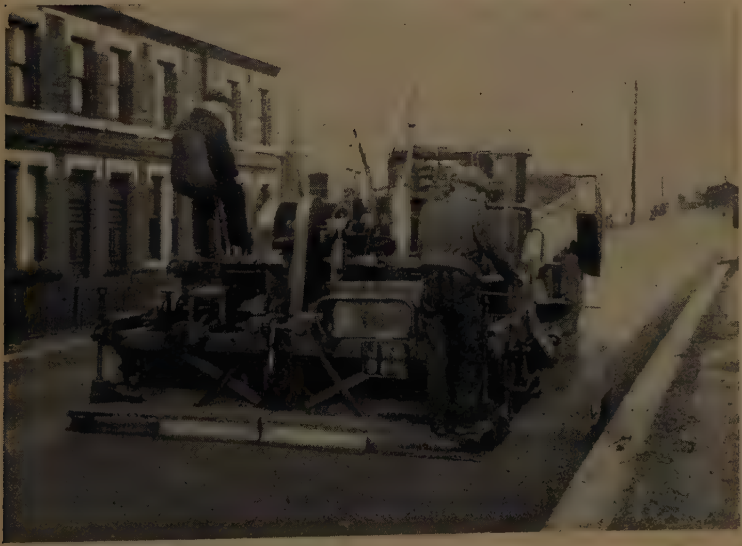
Close-up of Phoenix-Edwards Spreader showing Screed Plates and Manually Operated Adjustable Screeds, Levelling Devices, etc.

Fig. 11



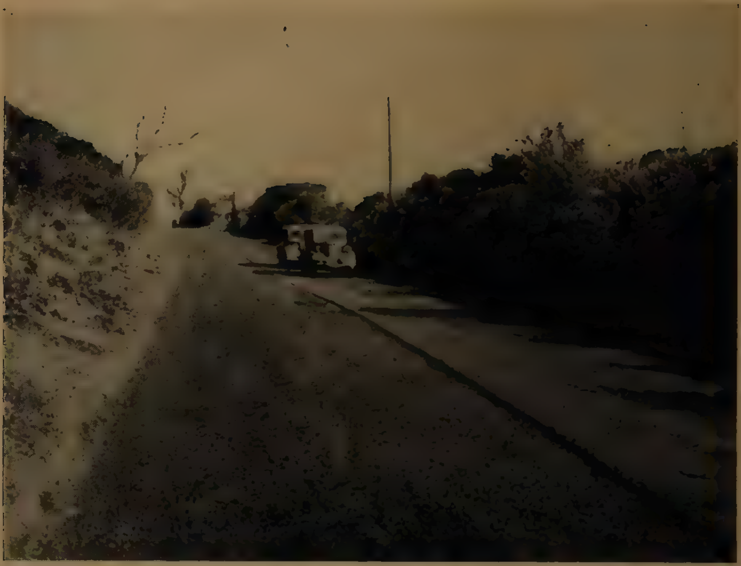
The Phoenix-Edwards Spreader in Operation

Fig. 12



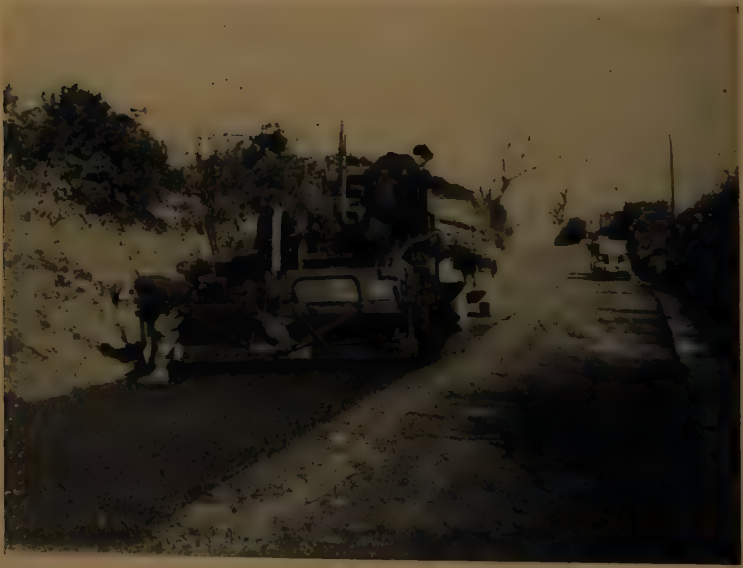
The Barber-Greene Machine in Use, showing Supply of Materials to Machine

Fig. 13



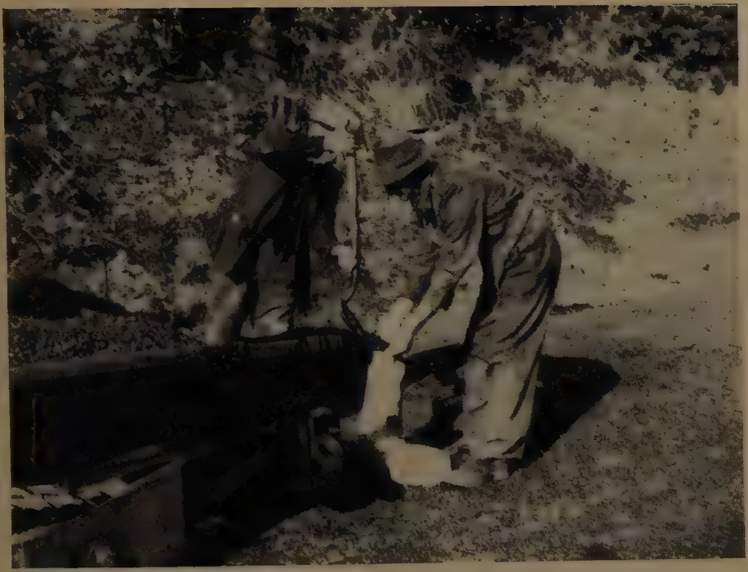
Half Width of Road Surface Laid by Barber-Greene Machine, showing Centre Joint left by Machine

Fig. 14



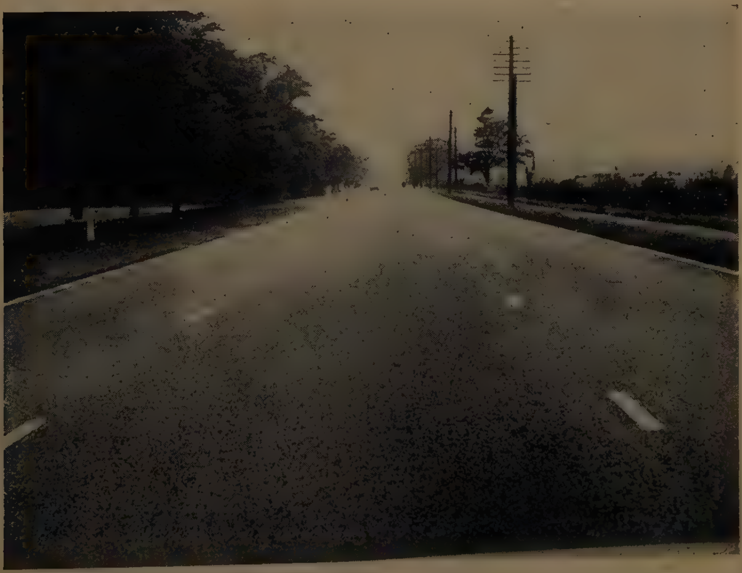
The Barber-Greene Machine in Operation, matching up Centre Joint shown in *Fig. 13*

Fig. 15



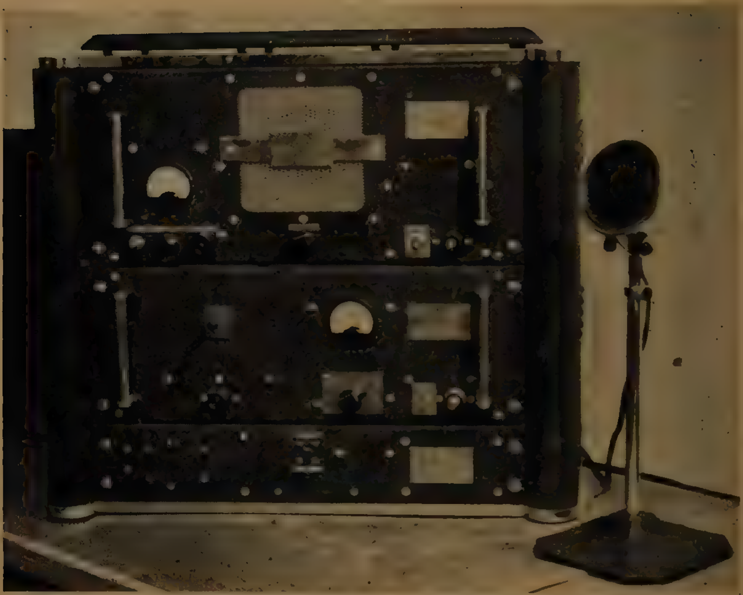
Taking Samples for Laboratory Analysis from Barber-Greene Machine

Fig. 16



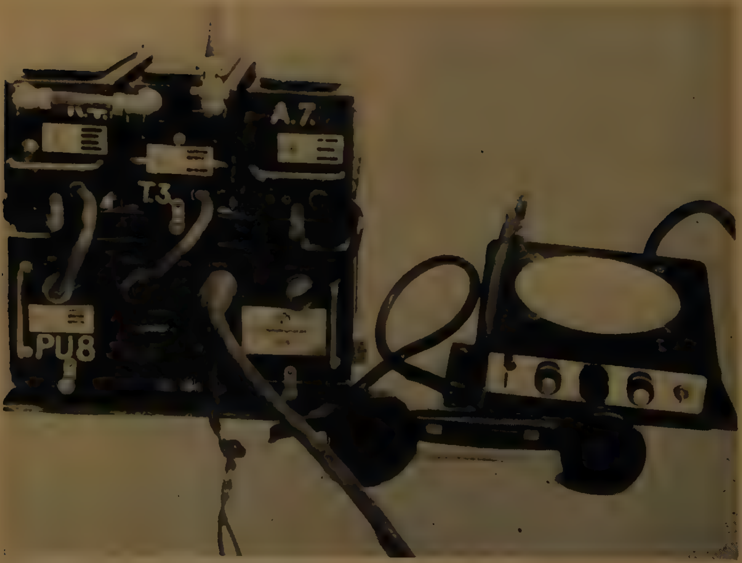
Finished Surface obtained with Barber-Greene Machine on Great North Road. Medium Texture Carpet

Fig. 17



Main Transmitting and Receiving Set

Fig. 18



Small Receiving and Transmitting Unit for Use on Work Sites or Vehicles strip, overlapping the first strip by a width of 1–3 inches to allow sufficient material to be forced into the joint by the finisher. The overlapping material, which cannot be compressed into the set material of the first strip, must be removed before the joint is rolled. Care should be taken when removing the overlap not to disturb the first strip nor to remove the material properly belonging to the second strip. The first pass of the roller on the second strip should be made either in the manner already described or adjacent to the longitudinal joint in order to prevent any lateral movement of the material away from the matched joint when it is rolled. The roller should be run over the joint as soon as possible behind the machine while the material is still workable. On rolling, in general, it should be emphasized that a machine-laid surface *must not* be rolled until the surfacing is firm enough to bear the roller without displacement.

Although it is desirable, whenever possible, to match the two strips before the material has set, in order to produce a joint which is not visible, this should not be done at the expense of laying short lengths and then changing over purposely to match the longitudinal joint. The disadvantage of not changing over quickly, which lies only in the joint being rather more visible than it would otherwise have been, is preferable to the surface irregularities which may be the result of unnecessary transverse joints and breaks in laying caused by changing over unnecessarily from one strip to the other. The exposed edge of the first strip should be trimmed to a neat vertical face and good alignment before the second strip is laid.

Joints should always be matched with the machine standard on the side of the matched joint—if a cut-off shoe is required then it should be placed on the opposite side of the machine.

To complete the end of a strip, the feeding of the machine should be stopped and the machine operated forward until the material, spread across its width, is used up. The machine can then be moved away and the mat squared off. If it is intended that the strip be opened to traffic before the machine is brought back to match the transverse joint then a run-off should be added to prevent traffic from damaging the finished surface by breaking down the edge and bumping along the mat. The method normally used is to square off the mat and place a length of timber the thickness of the compacted mat, across the width of the strip against the squared-off edge. The roller thus does not push down the end of the strip and feather the edge. The run-off should then taper from the rear edge of this timber for a reasonable length, dependent on the thickness of mat laid. Alternatively, if a transverse joint is to be made before the mixture has set, the roller should be kept off the last two yards of the strip. Then, to make the joint, the screed unit should be lowered on to the unrolled part of the mat and the thickness controls placed in the same position as when the strip was completed. The normal operations of feeding material and operating forward should be carried out and the joint made.

Transverse joints made from a compacted mat should first be longitudinally checked with a straight-edge and any irregularity removed by cutting back the joint edge until the straight edge fits correctly. The joint should then be completed in the same manner except that an allowance for roller compaction should be made. It should be emphasized that surface irregularities are prone to occur at transverse joints, and in order to achieve a high standard of finish it is advisable to keep the number of joints to the minimum.

Where it is possible to allow traffic to use part of the road while the machine is working, single-line traffic control should be rigidly enforced to prevent damage to, and displacement of, the surface or its edges before the mixture has set or been properly consolidated. If traffic is permitted to run at random over a freshly laid unmatched strip it breaks down or bevels the edge of the mat, thus destroying the straight abutment so necessary when matching a longitudinal joint. Some surfaces laid with mixtures of a friable nature can be spoiled by the passage of traffic before the mat has set or been rolled. The damage, in this case, generally consists of shallow ruts and the surface cannot be successfully reinstated by hand.

When laying on roads which are not wide enough to permit vehicular traffic to pass the machine when in operation, the road should be closed and a diversion route provided. This prevents interruption of supply, and disturbance of laying, and avoids the delay caused by frequently moving the machine to one side for traffic to pass, and eliminates any surface irregularities which may be formed by unnecessary transverse joints. On wide carriageways it is sometimes possible to cordon off the strip being machined, and permit the traffic to use the width remaining without need of control.

Defects in the finished surface can be readily observed and eliminated with proper attention. Tearing or dragging of the machined surface is the worse and commonest type and is an indication of improper operation or adjustment, or unsuitable material. Tearing can roughly be divided into three classes. The first kind of tear is usually caused by partially set material being delivered to the machine hopper after a long haul or delay. The cold lumps of mixture do not break easily and are dragged along by the screed unit, thus making a score or hole. Adequate sheeting and precautions against loss of temperature can minimize or eliminate these defects, as previously explained. Foreign objects, such as pieces of wood, metal, and rags sometimes get into the mix and cause holes in the surface as they are dragged along. Secondly, screed tears or dragging may be caused by a rough, dirty, or improperly adjusted screed. Such defects occur because the mixture sticks to the screed unit and causes dragging or scoring. The third class of tear appears as a series of cracks across the machined surface and is caused by a poor or dry mix. Instead of the mixture being compacted by the machine it drags forward slightly before striking off. This results in a series of cracks $\frac{3}{8}$ to $\frac{1}{2}$ inch apart and about $\frac{1}{2}$ inch deep.

All these defects can be eliminated by the remedies given, or prevented by correct operation, but the only immediate correction if they are found to occur is to make good the defective surface by filling in the holes or tears by hand spreading. Small tears or holes attributable to a poor mix can be closed by raking material into the area and rolling the surface, but large holes in the mat caused by the dragging of lumps and partially set material normally require more attention. The surrounding material should be loosened, the hole filled in to make a proper patch of the correct thickness and the surface re-rolled.

This practice should not be necessary when machine laying is correctly carried out and filling back should be avoided at all times. It is the surest method of producing surface irregularities.

Causes of minor surface irregularities and their remedies are numerous and they can best be illustrated by the operation chart for a Barber-Greene Finisher given in *Fig. 2*.

The transverse shape traditionally adopted for single carriageways draining to each side of the road is either segmental or an approximation to a parabola, the minimum transverse drainage gradient being provided at the crown of the road and becoming rather steeper at the sides so that water may be drained into the channels positively and quickly. The finisher cannot lay to this shape. Its screed, which is normally straight, can either be " broken " at the centre so as to give a versine of $1\frac{1}{2}$ inch, or tilted at either end 6 inches above its normal plane. Transverse shape must therefore be obtained in terms of these movements and dimensions.

When the surfacing is being laid in one strip, the camber must obviously be obtained by " breaking " the screed in the centre to give the necessary crossfall. When the surfacing consists, for example, of two strips, the method to be used will vary according to the type of material being laid. Plastic materials, which can be moulded slightly during rolling, may be laid with a straight crossfall from the crown to the centre of the road and the roller may be used to ease off the central peak. Harder non-plastic materials are better laid with a " broken " screed on each strip, the screed being tilted as well as " broken " so as to provide the minimum transverse gradient on the central half-strips.

During the course of a transitional length from balanced road to superelevation, the crown setting should be gradually removed throughout the length until the screed is straight at the start of full superelevation.

It is usually better to mark service covers and lay over them and to re-set after surfacing has been completed. If they are raised before the surfacing is laid it may sometimes be found that they come in the track of the crawlers, and interfere with levelling control. The machine cannot (and should not try to) lay the surfacing to the cover level. Small differences in level between the cover and the surfacing material will mar a good

Fig. 2

		Poor plant operation	Excessive moisture in mix	Lack of bitumen in mix	Too much bitumen in mix	Poorly graded mix	Poor bitumen product	Segregation of mix in lorries	Poor control of 200 mesh material	Poor temperature control of mix	Improper operation of finishing machine	Over-controlling of screed	Improper position of tamper bars
Checking under roller	W			X		X	X		X				
Water pockets or short low section in surface	V					X		X					
Poor levelling control of screed	U	X			X	X			X	X	X	X	
Unsatisfactory compaction	T	X		X			X		X	X			X
Bleeding	S				X	X							
Tamper marks	R										X		
Mat shoving ahead of roller	Q		X		X	X	X			X			
Roller marks	P												
Bad transverse joints	O										X		
Bad longitudinal joints	N					X				X	X	X	X
Wavy surface (short waves)	M									X	X		
Wavy surface (long waves)	L									X	X	X	
Rough uneven surface	K	X			X	X		X	X	X	X	X	X
Pock-marked surface	J												
Deep transverse cracks across full width of strip	I	X				X			X	X			X
Varying surface cracks on surface of strip	H	X				X			X	X	X		
Surface cracks at edges of strip	G	X				X			X	X	X		X
Surface cracks at centre of strip	F					X			X	X	X		X
Transverse hair cracks in surface	E									X	X		X
Brown streaked surface	D					X	X		X				
Rich shiny appearance of surface	C	X			X	X	X		X	X			
Lean brown dead appearance of surface	B	X	X	X		X	X		X	X			
Fluctuating surface texture	A	X				X	X	X	X	X			

		End section of of screw conveyor disengaged	Agitator should be installed	Crawler drive clutches slipping	Crawler take-up too tight	Tamper not up to speed	Finisher travelling too fast	Loading screw conveyor too heavy	Worn out screed plate	Worn out tamper bars	Screed rests on surface while stopped	Allowing roller to stand on fresh surface	Putting roller on fresh surface too soon
Checking under roller	W												
Water pockets or short low section in surface	V										X	X	X
Poor levelling control of screed	U		X					X					
Unsatisfactory compaction	T					X	X			X			
Bleeding	S												
Tamper marks	R					X	X			X			
Mat shoving ahead of roller	Q						X						X
Roller marks	P											X	X
Bad transverse joints	O												
Bad longitudinal joints	N			X			X		X	X			
Wavy surface (short waves)	M				X								
Wavy surface (long waves)	L		X					X					X
Rough uneven surface	K		X				X	X	X	X	X	X	X
Pock-marked surface	J												
Deep transverse cracks across full width of strip	I		X				X						
Varying surface cracks on surface of strip	H	X							X	X			X
Surface cracks at edges of strip	G	X							X	X			
Surface cracks at centre of strip	F								X	X			
Transverse hair cracks in surface	E												X
Brown streaked surface	D												
Rich shiny appearance of surface	C												
Lean brown dead appearance of surface	B												
Fluctuating surface texture	A												

		Rolling too fast	Reversing roller too abruptly	Water not used on rollers	Turning roller too abruptly	Too much slack in roller drives	Roller too heavy	Three-wheel roller used instead of tandem	Roller too light	Prime or tack coat not used	Dusty or dirty base	Prime or tack coat applied too heavy	Inadequate rolling
Checking under roller	W	X	X		X		X			X	X	X	
Water pockets or short low section in surface	V		X		X	X		X					X
Poor levelling control of screed	U												
Unsatisfactory compaction	T	X							X				X
Bleeding	S											X	
Tamper marks	R												
Mat shoving ahead of roller	Q		X				X			X	X		
Roller marks	P	X	X	X	X	X	X	X					X
Bad transverse joints	O												
Bad longitudinal joints	N	X		X					X				
Wavy surface (short waves)	M					X							
Wavy surface (long waves)	L	X											
Rough uneven surface	K	X	X	X	X	X	X						X
Pock-marked surface	J												
Deep transverse cracks across full width of strip	I												
Varying surface cracks on surface of strip	H	X			X	X							X
Surface cracks at edges of strip	G												
Surface cracks at centre of strip	F												
Transverse hair cracks in surface	E	X				X							X
Brown streaked surface	D												
Rich shiny appearance of surface	C												
Lean brown dead appearance of surface	B												
Fluctuating surface texture	A												

		Too much rolling	Mix not suited for cold weather application	Mix not suited for hot weather application	Unstable mix	Soft stone or clay balls in mix	Petrol, paraffin, oil spilled on surface	Soft base	Poorly graded base surface	Excessive length of strip	Not rolling joint soon enough	Screed tension spring too tight	Hydraulic jack not released	Brakes set on lorry
Checking under roller	W	X			X			X						
Water pockets or short low section in surface	V	X			X			X	X		X			
Poor levelling control of screed	U				X				X			X	X	
Unsatisfactory compaction	T		X						X			X	X	
Bleeding	S			X										
Tamper marks	R		X											
Mat shoving ahead of roller	Q			X	X									
Roller marks	P	X			X	X								
Bad transverse joints	O									X	X			
Bad longitudinal joints	N		X	X	X	X				X	X			
Wavy surface (short waves)	M				X	X								X
Wavy surface (long waves)	L			X		X						X	X	X
Rough uneven surface	K	X	X	X	X			X	X			X	X	X
Pock-marked surface	J					X	X							
Deep transverse cracks across full width of strip	I		X											
Varying surface cracks on surface of strip	H		X		X									
Surface cracks at edges of strip	G													
Surface cracks at centre of strip	F										X			
Transverse hair cracks in surface	E		X		X						X			
Brown streaked surface	D						X							
Rich shiny appearance of surface	C													
Lean brown dead appearance of surface	B													
Fluctuating surface texture	A													

Fault-Finding Chart of Barber-Greene Finishers running surface, and it is important that the covers should be correctly matched into the road surface.

INTERCOMMUNICATION

In order that the maximum degree of efficiency may be derived from a fully mechanized organization, it is essential that a suitable and simple method of intercommunication should be available for the purpose of rapid direction and constant control of the various operations, especially when divorced from the normal telephone service. It has been found that the radio telephone communication system is admirably suitable for these forms of mobile operations and, in addition, can be used with considerable advantage for the direct control of snow-clearing equipment during winter emergencies, thus enabling the radio equipment to be continuously employed throughout the year with resulting benefit to the maintenance of the radio equipment and operating personnel.

A radio communication system is in use in Lanarkshire and comprises a very-high-frequency simplex single-frequency radio telephone, of which the carrier wave is amplitude-modulated. This system is extremely flexible for providing communication between any number of mobile units and one or more fixed points. The mobile equipment can be mounted in a car, lorry, roller, trailer, snowplough, or any such vehicle where a 6- or 12-volt battery can be made available to provide the power to operate it. The fixed, or static, stations operate from the AC mains supply and can be either directly or remotely controlled.

Each set works on the single fixed frequency of 86·425 megacycles per second and, since the frequency is quartz-crystal controlled, no tuning adjustment is necessary during operation, because the sets are always automatically tuned, and the equipment is extremely simple to use. With this system of simplex operation between any two stations only one operator can speak at a time since, when one is transmitting, his receiver is automatically switched off. No skilled personnel are therefore required to operate the equipment as the method of operation is almost similar to that of the ordinary Post Office telephone.

The effective range of the equipment largely depends upon the height of the aerials and the nature of the intervening terrain. Over flat country the range is about 25 per cent greater than the optical, or line-of-sight, range, whereas the range obtainable in mountainous country, or within heavily built-up areas, may be correspondingly less, owing to absorption by the hills, buildings, etc.; so the siting of the fixed stations is an important factor if maximum effective range is to be obtained. The average range of the fixed stations is 25 miles radius with a radio-frequency power output of 12 watts. However, with simplex single-frequency working, the mobile stations can relay messages to and from the fixed stations, or communicate directly with each other, when operating in difficult country, and thus facilitate combined operation and enlarging the effective range.

MAINTENANCE

As mentioned in the introduction, only the maintenance of the Barber-Greene machine is dealt with in this Paper, and the following information has been obtained from Messrs E. L. Harber and F. G. Flockhart of Jack Olding and Co. Ltd, the suppliers of the Barber-Greene machines in Great Britain.

In brief general instructions on maintenance they state the following, gained as an accumulation of knowledge during manufacture and operation in the field :

> "From experience we find that with few exceptions Contracting firms expect the operator to do his own maintenance whereas in most cases the County Authorities arrange for maintenance staff over and above the operator, and they do definitely allow both Saturday morning and a number of hours each day to be set aside for maintaining the machine, whereas the operator who is also responsible for maintenance has invariably to carry out his maintenance either prior to or after the days working, or alternatively, between loads.
>
> It must be realized that the average operator is not going to do much maintenance before or after loads which, in effect, is before or after his normal day's working. Very few machines, if any, are working to anywhere near capacity, and there are often periods during the day's working when there are considerable time lags between loads. Even a small amount of maintenance during these periods will help to eliminate unnecessary breakdowns, due to lack of maintenance."

The remainder of the information they have supplied is given in the Appendix.

CONCLUSION

The Authors hope that what has been written here affords a further demonstration of what Telford said in his memorandum to the House of Commons in 1818—"from this it will be seen that a much higher degree of skill is required in roadmaking than is commonly supposed to be the case."

ACKNOWLEDGEMENTS

Mr E. W. W. Richards wishes to acknowledge his indebtedness to the Highway Engineering Educational Trust and the Worshipful Company

of Paviors for the opportunity of acquiring much of the information contained in his contribution to this Paper, and for permission to publish it. He also wishes to thank the Commissioner, Public Roads Administration, Washington, for permission to publish the American statistical data and the Barber-Greene Co. of America for extracts from their publications.

The Paper is accompanied by sixteen photographs and two sheets of diagrams, from which the half tone page plates and Figures in the text have been prepared, and by the following Appendix.

APPENDIX

Messrs Harber and Flockhart's observations so far as cleaning, adjustment and maintenance are concerned follow very closely those given in the manuals supplied by the makers—"Barber-Greene 879A Finisher, Parts Catalogue and Maintenance Instructions" and "879A Finisher, Parts Catalogue, English Supplement."

On lubrication they make the following remarks:

Nothing can add more to the life of the machine than thorough lubrication of moving parts, but it must be remembered that life is only added by correctly applied lubrication in accordance with manufacturer's instructions. Mis-applied lubrication can cause troubles equal to those caused by no lubrication whatsoever.

In the case of roller or ball bearings, there is a rubbing between the cage and the rollers and it is this rubbing which calls for the application of a suitable lubricant. Here it must be remembered that the lubricant, used to minimize this rubbing, itself sets up a degree of resistance to movement however thin it may be. The lubricant also acts as a preservative of the finely finished surface of the ball or roller bearing against attacks of moisture. Generally speaking, the steel from which bearings are made is particularly susceptible to outside influences such as moisture, dust and grit.

"Grease" is a name descriptive of a great variety of materials, many of which are totally unsuitable for application either to ball or roller bearings, and it is necessary to select the grease to be used with the greatest care.

There are basically two types, lime base and soda base greases. The lime base grease is softest in consistency and tends to settle in the bottom of the bearing where, if excessively churned by high peripheral speeds, i.e. speeds over 600 r.p.m., it will become overheated.

As the shaft speed carrying the tamper eccentric and jack-knife bearings is double the speed quoted, the grease required must be a soda base to conform with the following recommendations:

Liberty HMP Grease
Shell V.W.
Vacuum B.R.B.3.
Duckhams Z.3343.
Price R.B.
J. Kerr
Wakefields Spheerol S.
Texas Oil Co. Regal Starfak No. 2.

The general instructions for lubricating the various points are as follows

(a) *Main Transmission*

Check lubricant level every 50 hours. Change every 200 hours the level plugs at hopper end. Two level plugs, fill to the top one which is the level.

(b) *Tamper Clutch*

Two grease fittings. Grease every eight hours. Also see that the levers, pins and rollers are lubricated from time to time. The clam should be lubricated frequently and also the gunmetal bush in the pulley.

(c) *Gear Box*
Check lubricant level every 50 hours. Change every 200 hours. Remove inspection port cover to fill. Pet cock for level reached through face of machine.

(d) *Crawler Intermediate Shaft*
Four hydraulic fittings. One shaft each side of machine with two bearings on each shaft. Grease every eight hours. Every two hours if travelling extensively. Open side covers for two fittings. One fitting piped to top at tamper clutch. Remove cover on hopper side of engine for other fitting.

(e) *Crawler Spring Idler*
Two hydraulic fittings, one each side of machine. Grease every eight hours. Every two hours if travelling extensively.

(f) *Levelling Arm Pivot Bearing*
Two hydraulic fittings, one each side of machine. Grease every eight hours.

(g) *Pan Feeder Foot Shaft*
Four hydraulic fittings. One shaft each side of machine with two fittings in end of each. Grease every eight hours. Lift flap over bar feeders.

(h) *Truck Rollers*
Two hydraulic fittings. One each side in end of roller shaft. Grease every eight hours.

(i) *Truck Rollers*
Clean with paraffin or fuel oil to keep turning free. Check every eight hours.

(j) *Crawler Foot Shaft*
Two hydraulic fittings. One shaft each side of machine with fitting in end. Grease every eight hours. Every two hours if travelling extensively.

(k) *Crawler Rollers*
Twelve hydraulic fittings. Six each side of machine. Grease every eight hours. Every two hours if travelling extensively.

(l) *Crawler Drive Shaft*
Two hydraulic fittings. One each side of machine. Grease every eight hours. Every two hours if travelling extensively.

(m) *Pan Feeder Head Shaft*
Four hydraulic fittings. One shaft each side of machine with two bearings on each shaft. Grease every eight hours from below. Run machine up on six inch blocks.

(n) *Screw Conveyor Bearing*
Eight hydraulic fittings piped to top. Grease every four hours. Use high temperature grease when handling hot mixes.

(o) *Screed*
Clean bottom of screed with kerosene or fuel oil. Check every eight hours. Paint with heavy oil to prevent rust when machine is not to be used for over a day.

(p) *Thickness Control Screw Bearings*
Two hydraulic fittings, one each side of machine. Grease every eight hours.

(q) *Thickness Control Screw Nut*
Two hydraulic fittings, one each side of machine. Grease every eight hours.

(r) *Crown Control Thrust Bearings*
Two bearings located at top centre of screed. Pack with grease when overhauling. Oil with motor oil every eight hours.

(s) *Tamper Links*
Twelve hydraulic fittings. Grease every four hours.

(t) *Screw Conveyor Countershaft*
Two hydraulic fittings. Grease every four hours.

(u) *Feeder and Screw Conveyor Clutch Shaft*
Located in gearbox. Two hydraulic fittings on bearings. Two hydraulic fittings on bushed sprockets—avoid too much grease. Two hydraulic fittings on clutch shifter rings. Grease every eight hours. Raise hinged covers.

(v) *Feeder and Screw Conveyor Countershaft*

Four hydraulic fittings, two each side in bearings. Grease every eight hours. Raise hinged covers.

(w) *Gear Box Countershaft*

Two hydraulic fittings on bearings on each side of gear box. Grease every eight hours. Raise hinged covers.

(x) *Gear Box Crawler Clutch Shaft*

Two hydraulic fittings on bearings. Two hydraulic fittings on bushed sprockets —avoid too much grease. Two hydraulic fittings on clutch shifter rings. Grease every eight hours. Raise hinged covers.

(y) *Blower Drive Bracket*

There is one hydraulic fitting. Grease every eight hours with ordinary high pressure grease.

The following individual Units require special attention :

Blower

The Blower itself requires lubricating with engine oil every 25 hours, and oil can be applied by removal of one of the studs from the gear housing cover. After removal of the gear housing cover it will be seen that there is an oil reservoir running right through the main body of the casting to the bearings at the pump end, so that lubrication applied through the gear housing cover will automatically feed through and lubricate the opposite end of the pump body.

Tamper Eccentric and Jack-knife Bearings

These require very special attention and the grease recommendations made earlier were arrived at after consultation with the bearing manufacturer and the oil companies and should be strictly adhered to.

Previous to this recommendation troubles were experienced with over heating which was mainly due to the wrong type of lubricant being used, and too much being applied.

It is important with the comparatively high operating speeds that the bearing housings should not be tightly packed with grease. The recommendation is that the bearing itself be completely packed and the housing space only half filled allowing the grease, when the machine starts up, to leave the bearing and enter the space remaining in the housing thus preventing packing and churning.

The method of applying the grease is to remove one of the end covers and force the grease into the bearing by hand from one side allowing the grease to go right through the bearing until it appears on the other side. This application applies to both the tamper eccentric bearings and tamper plummer block bearings and also tamper jack-knife bearings. This application dispenses with the use of grease nipples, thereby eliminating the possibility of excessive grease being applied to the bearing. New grease should be applied every six months and thoroughly clean out all existing grease before re-filling.

Track Rollers

These require lubrication every eight hours, and special attention should be given to the two rear rollers due to these wearing faster than the others, this being caused by the weight of the screed on the rear end of the machine.

Whilst excessive travelling from site to site by the machine under its own power is not recommended it may be necessary, due to absence of transport or other reasons, to cover fairly long travel periods. Should this occur, special attention should be given to the track rollers on route, and lubrication applied every two hours.

Discussion

Mr Lovell, in presenting the Paper, explained how the Paper had come to be written. He had been given nominal responsibility for writing it, and, in covering the subject, he had tried to get experience from the north, from Mr Wilson, from the south, from Mr Richards, and from himself a representing the middle of England. In the Paper there were a goo many data gleaned from American sources, during a visit which M Richards had made to America and were definitely his contribution t the Paper, a contribution which Mr Lovell found of extraordinary interest

The object of heating and planing was to overcome irregularities on roads with too fat a surface. It was not applicable to, or generally problem on, anything except a road modernized in the past decade and subject to heavy traffic. It was intended to deal with surface irregularities and was no good for foundation troubles. It was particularly useful in built-up areas, but in that connexion he ought to give the warning that danger and nuisance from smoke might on some occasions be very real

The planed-off material could be used again, and that in itself was very often an asset, because it made the operation overall cheaper to that extent. He would like to pay tribute to the trade for the development of the plant which was used. They had done all the development work and the plant was the result of their direct interest in the matter. There were two main types of machine, those with V-blades and whose with reciprocating blades or rotary blades. There was also the grader, which could be used as a planing machine though not specially designed for that purpose.

In the Paper some prices had been given, but the figures were a little on the high side, and prices could soon be expected to be a little lower than those quoted.

There was also the necessity of subsequently treating the surface of a road after it had been planed. It was not safe to leave it indefinitely, or very often the seal was broken and trouble occurred through water getting into the foundations.

The next section of the Paper dealt with non-power-operated spreaders. That was a half-way house between hand spreading and machine laying. Such a machine was useful for small jobs and narrow roads. Its efficiency depended primarily, in his opinion, on the use of a suitable material, which was generally of the cold asphalt type. Its principle involved a series of screeds, and by the material flowing across the road under the forward motion of the screed a level and uniform surface was obtained. It required intelligent feeding and towing, and there would be trouble if it were towed by a lorry and the lorry were allowed to move forward with a jerky motion. Too much should not be expected from the machine; it could not replace a Barber-Greene, but when used in addition to the Barber-Greene it could be a very useful piece of plant.

He proposed to leave the whole question of power-operated finishers to be dealt with by Colonel Richards, and to turn to the section on intercommunication towards the end of the Paper. That was often a sadly neglected piece of organization, and very often only necessity and costs forced the issue by raising their ugly head. In that connexion he would draw particular attention to the transport-movements chart, *Fig. 1*, which illustrated the point which he was making. To refer to some of the difficulties which were experienced if intercommunication was not working properly, the Barber-Greene machine might break down and lorries would be on the way, and probably the material coming in was wasted. In bad weather, again, the material might be wasted. On the other hand, the quarry plant might break down and there would be a gang on the road and the plant waiting, and that would put up costs and often led to the criticism that people were standing about and no work was going on, and so on. To his mind, radio was the obvious answer. It was extensively used in private practice, and its development for work of the kind in question was long overdue. In local government use, a point to be decided was the hook-up desirable, whether it should be on the network of the highway department or on that of the police or ambulance service, by some arrangement with them. It was a question to be decided—whether the highway department's own network was justified or whether there should be joint use of as much as practicable of the facilities available. Further thought would have to be given to that.

The last section of the Paper dealt with maintenance, and he would like to thank Messrs Harber and Flockhart, of Jack Olding and Co., for much of the information contained in the Paper. None of the information in the rest of the Paper was of any use at all if maintenance was neglected. To do its job the machine had to be in perfect working order, and for it to be in perfect working order adequate maintenance was, of course, essential. The intelligent reading of any report on a large civil engineering job showed that systematic and regular maintenance paid dividends, but maintenance, unfortunately, was often taken for granted.

Mr Richards, continuing the introduction to the Paper, said that the substance of the rather lengthy section on power-operated finishers could be expressed in two short statements :—

(1) That to produce the good finished road surface which the machine was capable of laying it had to be operated with a knowledge of the principles of its construction and of the characteristics of the materials which it was laying.

(2) That high output and low costs could not be achieved without close attention to organization and the development of surfacing materials suitable for fast laying.

It might be thought, from traditional conceptions, that those statements were obvious but incompatible.

No one would dispute the validity of the statement that correct operation of the finisher and supervision appropriate to its rate of expenditure were important, and that it would make a great difference to output, cost, and quality.

It had been stated[1] that "good standards of evenness have frequently been achieved over lengths of 10, 20, or 30 feet, standards which were much better than the average for the road. Yet what was obtained over 20 feet should, with similar skill and attention, have been possible over 200 feet, or 2,000 feet; relaxation of attention in one direction or other must be presumed to have been the cause of the frequent lapses from a standard which was demonstrably attainable."

The object of that part of the present Paper which dealt with the operation of the finisher was not to provide a manual for operators but to demonstrate the principles on which correct operation depended so that all surfacing work might be raised to a uniform standard of excellence. The finisher itself should play a major, if not the predominant part, in the surfacing operation and it might often be desirable to change other factors in the surfacing operation to suit the technique of laying by finisher rather than, as was so frequently done, to slow down the pace of the machine to suit a surfacing material which might possess no compensating advantages.

In Hampshire, the information given by Mr Scott's Paper,[1] supported by some local work, had led to the conclusion that the laying of surfacing materials on unprepared bases—sound but irregular in shape—was a practical proposition.

It was undoubtedly true that the ideal road surface was produced by laying upon a smooth and even base, of uniform strength throughout, a surface strong enough to be free from any tendency to disintegrate and stable enough to resist deformation under traffic. That state of affairs seldom, if ever, occurred and the road engineer was usually faced, so far as the base was concerned, with the problem of eliminating irregularities either before or when applying his final surface. If he desired to lay a uniform thickness of material he might do it by two methods of equalization—the first being to plane or scarify and roll the foundations to an even contour, and the second to fill in the depressions with some form of surfacing material before laying the surface coat proper. Both of those methods had their disadvantages. The first involved disturbing or tearing up a foundation consolidated by many years of traffic compaction. The rolled surface could never be as hard as the portions which had not been scarified, and the result was that the base, of uneven bearing value, contained within itself the seeds of future settlement and irregularity. The second of those methods—to fill in the hollows beforehand—could be of value only if it was possible to lay a material which had greater mechanical stability than that which was being used for the surface. That was seldom possible, and could be done only where the irregularities were of a large order. A frequent practice was to make up irregularities by using a finer material than that of the surfacing coat. That finer material usually had less mechanical stability than the surfacing itself, and could be justified only on the grounds of convenience and expediency in the days of machine-laid and compacted surfaces, though it had undoubtedly been of great value in the days when surfaces had been laid by hand and there had been a strong, though perhaps unconscious, tendency on the part of the laying gang to follow the irregularities of the base, or when differential compaction had been likely.

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[1] W. J. O. Scott, "Roads and their Riding Qualities." Road Paper No. 25, Instn Civ. Engrs, 1948.

Now that most, if not all, surfacing work could be undertaken by the Barber-Greene Finisher—which produced a smooth and regular top surface with uniform compaction—the surfacing and compensating courses might, within limits, be laid in one operation. Before the finisher had been available there had been very real objections to that procedure, though it had undoubtedly occurred, and with little harm, to a much greater degree than was commonly supposed. Now that the finisher (and especially the compacted-strike-off type) was available many of those reasons had disappeared and the balance of advantage had changed to the extent that there should not be positive reasons for rejecting so logical and economical a procedure within the limits where it might be applied.

In Hampshire, in the year 1946–47, before the present practice had been adopted, approximately 44 miles of road had been surfaced, of which only 8 miles had been laid by finisher. In the following year (the year of partial conversion), 108 miles of surfacing had been laid. In 1948–49, the first full year of operation under the new system, 183 miles of road had been resurfaced (160 of them by Barber-Greene) at the same cost as the 108 miles laid in the previous year.

Those figures were not unaffected by other considerations but it should not be thought that that expansion had been effected by spreading the material more thinly, and so covering a greater area; indeed, more than 50 per cent of that tonnage had been more than 3 to 4 inches in thickness, more than 75 per cent was greater than 2 to 3 inches in thickness, and more than 85 per cent of it greater than 1½ inch thick.

In the year in which the bulk of the money had been spent in securing the existing road surfaces against deterioration by means of resurfacing, it had been estimated that the value of that practice, on the basis of the costs of the method which had been superseded, was nearly half a million pounds a year. In Hampshire the change of practice had been a calculated risk, justifiable only in terms of the necessity which had existed at that time. The risk had been well taken; the surfaces were still sound and undeformed after some years. The practice was now so widespread that within a few years it had become a commonplace. That result could not have been achieved if the technique had not been adapted to the mechanical devices then available.

Turning to the second of his summarized statements, namely that high output and low costs could not be achieved without close attention to organization and the development of surfacing materials suitable for fast laying, Mr Richards went on to say that the Paper itself contained sufficient illustrative detail to emphasize the point that close attention to organization and supervision would not be wasted. Present differences between theoretical capacity and actual laying rates, and between British and American rates of progress, were sufficiently wide to make it evident that improvement was possible. Momentary laying rates of a high order had been achieved on roads in Great Britain, but there was still much to be done in that field. Engineers and research workers in Britain had tried for a long time, and without much success, to devise and specify surfacing materials from first principles, but as yet no satisfactory correlation had been found between the standard tests prescribed for the materials of which surfacing mixtures were made and their behaviour on the road; in fact, the only good mixtures were those which had been tried and found to be so. It had been suggested in the Paper that the problem might be tackled from the other end by relating the characteristics of the finished surfacing material to its behaviour on the road. There was already a substantial American background for that idea, and tests and standards already existed for its application.

In any examination of the subject Mr Richards felt that it was essential that Britain, as a nation, should make some effort to get away from a retrospective adherence to the materials which were still specified in the British Standards for lack of experience in machine-laying methods. He appreciated that the proper place for that to be done was during the drafting of the Standards themselves, and he hoped that ultimately it would be done there. If it were not soon accepted that the machine was to play an important if not predominant part in surfacing technique, then there would be no escape from the uneconomic practice by which the quantity of material which would be laid in a normal working day in Great Britain was limited to 100 to 250 tons as compared with the 800 to 1,200 tons which was normal for the United States. Whilst saying that, Mr Richards would not countenance the idea that fast laying and high quality were incompatible.

It could of course be argued that the British highway engineer was at present too poor to be efficient or economical, and that though improvements might be made in technique and organization he might be completely frustrated by the size of the job itself. A proper integration of plant, material, and job was obviously essential before high output and low costs could be achieved, but nevertheless, even under present conditions there would seem to be room for improvement in organization and the use of equipment, and little increase in output or reduction in costs was to be expected until those matters had been attended to.

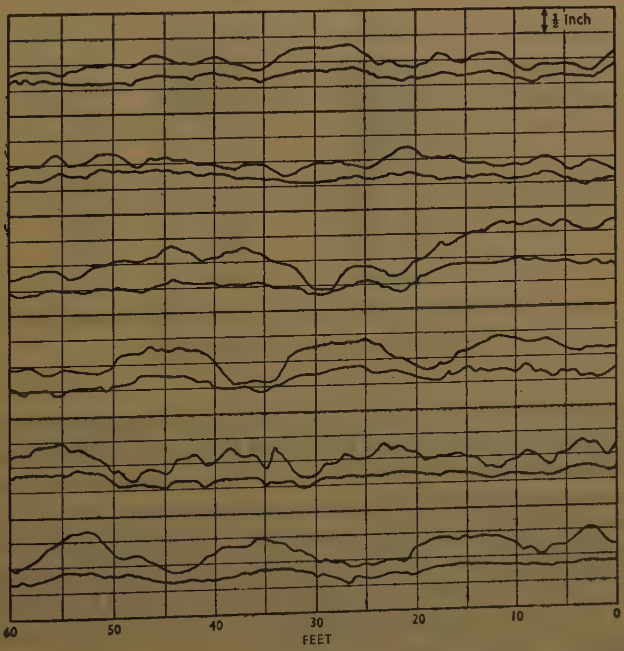
Many of the statements and arguments advanced in the Paper had far-reaching implications; some were of a controversial nature. They would not—indeed, should not—be accepted without examination and discussion.

The Chairman, when proposing the vote of thanks to the Authors, said that he was grateful to them for their Paper and to the Barber-Greene machine for adding a new interest to his life. He well remembered, in the early days, having had considerable difficulties with the older members of his staff when he insisted on the Barber-Greene machine being used for unprepared surfaces, and it was only when he said " It will be so " that the arguments ceased. If the Paper achieved one thing only—he was sure that it had done a great deal more—and made people realize that the Barber-Greene machine would not lay surfaces by itself, it would have done a tremendous amount of good. He had seen some bad work carried out with the Barber-Greene in Great Britain which he would have taken up. That arose from the fact that it had not been realized that the Barber-Greene would not work by itself but must be looked after and run properly.

The Authors had incorporated a tremendous amount of useful material in their Paper and were to be thanked not only for a valuable Paper but for the very considerable amount of time which they must have given to its preparation.

Dr W. H. Glanville observed that the Authors had described one aspect of the revolution which had taken place in road construction in the

Fig. 19



Longitudinal Profiles of Typical Surfaces before and after Heating and Planing-off Old Surface Dressings past decade. It was important that engineers engaged in the work should have the opportunity of describing their experiences.

Dr Glanville showed some slides illustrating the research work carried out at the Road Research Laboratory by Mr Scott, which Mr Richards had mentioned, and said that it would be useful for them to appear in the records together with the present Paper.

The curves in *Fig. 19* were in pairs, and represented the profiles of road surfaces before and after the heating and planing of the old surface dressing. The upper curve of each pair represented the original surface and the lower one the surface after planing. The intervals shown on the slide were ½ inch, giving some idea of the improvements, which were, in fact, considerable. The general form of the irregularity was carried through from the original surface to the remaining surface; the large waves were carried through to some extent.

TABLE 7.—EXAMPLES OF SURFACE IRREGULARITY MEASUREMENTS BEFORE AND AFTER HEATING AND PLANING

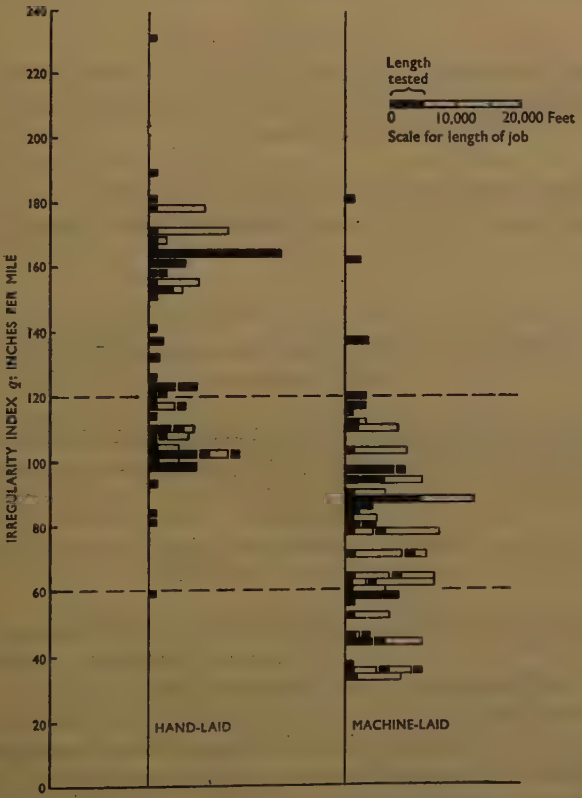
Machine	Irregularity Index, q: inches per mile		Reduction of Irregularity (per cent)
	Before	After	
Machine A	161	114	29
	143	87	39
	179	74	59
Machine B	236	69	71
Machine C	247	159	36
	143	90	37
	170	114	33
Machine D	145	88	39
	157	81	48
	169	82	51
	160	87	46
	150	94	37

(All sites more than 500 feet long)

Table 7 summarized those results and showed the irregularities in inches per mile, giving the total rise per mile of all the bumps added together for different heating-and-planing machines. It would be seen that after heating and planing, some of those surfaces had irregularity index values which were quite good—for example 74, 81, 82, and 90. Those were quite reasonable after planing, without any further surfacing. On p. 665 of the Paper the Authors said that "The heated and planed surface should be either surface-dressed or resurfaced within a few months." There were cases, of course, where that was essential, and perhaps one of the Authors would say something about them, but there were also cases where the evenness was sufficient and the condition was satisfactory for use for a number of years. Dr Glanville knew of one case where a road had continued in use satisfactorily for as long as 7 years without resurfacing. Very considerable economies, therefore, were on occasion possible by allowing the surface to remain in its planed condition. It was not an ideal surface, but it was much better than the original, for the motorist at least.

Fig. 20 had appeared in Scott's Paper and compared the inches per mile, the irregularity index, for hand-laid and machine-laid surfaces. It would be seen how much lower the machine-laid surfaces were as a group than the hand-laid surfaces. It had been found, for example, after the war that machine-laid surfaces were getting irregularity indices below 60, which was something almost unknown before the war with hand-laid methods. On the other hand, as the Chairman had pointed out, machine-laid surfaces could be bad if the machine was not properly used. The number of

Fig. 20

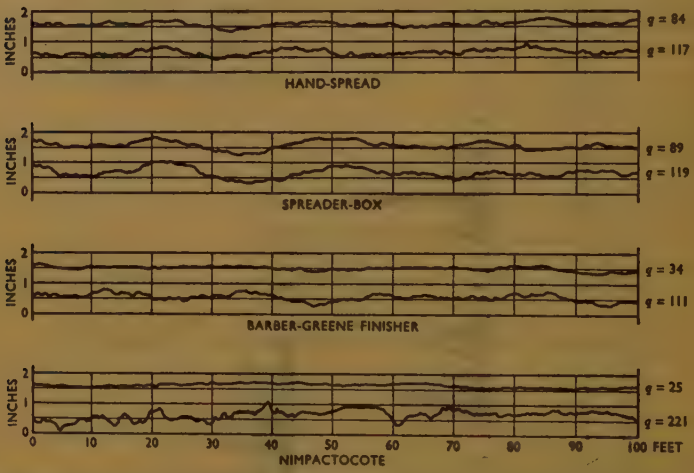


Surface Irregularity Data for Bituminous Surfacings occasions on which a machine was misused, however, was decreasing very rapidly as more experience was obtained.

Figs 21 compared different methods of laying bituminous surfacings and the effect of the surfacing on the irregularity. The upper curve was the finished surface and the lower curve the original surface. It would be seen from the third pair from the top how excellent a surface the Barber-Greene finisher had provided.

In the last paragraph on p. 679 the Authors had made a comparison between what they called "academic research" and what they called "production and cost studies." He had always found it difficult to understand what, in that field of applied research, "academic research" was; would the Authors explain what they had in mind?

Figs 21



COMPARISON OF THE DIFFERENT METHODS OF LAYING BITUMINOUS SURFACINGS (COLD-LAID FINE-TEXTURED ASPHALT)

It was very important to keep the question of machines in perspective and to relate it to the advantages to be obtained from lengthening the life of surfacing by paying careful attention to its composition. If half the cost of laying the surfacing were saved, it meant exactly the same saving, subject to some slight adjustment, as would be made by doubling the life of the material. There were two ways of economizing: one was by increasing the life of the material and the other by reducing the cost of production. He doubted whether as much as half the cost could easily be eliminated, but he had no doubt whatever that it was possible, by exercising care in production, to double the life of many of the surfacings which were put down to-day.

On p. 679 of the Paper the Authors referred to "qualitative tests," "desired characteristics," and to tests developed in the U.S.A. He would like some more information on that point also, if the Authors could provide it, because, although he kept fairly closely in touch with what the American engineers were doing, he did not think that as a whole they were satisfied with the methods of test which had been developed in America. Some of them were satisfied, and those who had developed tests in certain States seemed to be enthusiastic, but they seemed to become enthusiastic and to lose their enthusiasm much more rapidly than did engineers in Great Britain, and he did not think that many of them seriously felt satisfied at the moment that they had solved those problems.

On the question of intercommunication, he said that the scheme was an excellent one, because it was clear that all the methods in a mechanized train should be equally efficient, and intercommunication was one of the important elements in the train. He wondered, however, whether the radius of 25 miles with a power output of 12 watts was correct, because as a result of experience and from the Post office recommendations he would have expected the range to be much less than that for a power output of that order.

Finally, he would like to congratulate the Authors on their conclusion. Having written a very interesting Paper, they had said at the end of it:

> "The Authors hope that what has been written here affords a further demonstration of what Telford said in his memorandum to the House of Commons in 1818—'from this it will be seen that a much higher degree of skill is required in roadmaking than is commonly supposed to be the case'."

They must have enjoyed writing that; Dr Glanville knew that he would have done so. It was undoubtedly true, but it would have been very interesting to compare the "this" of those days with the "this" of to-day. He had never had the pleasure of reading that memorandum, but he had no doubt that Telford would have been very surprised if he could have read the present Paper.

Mr E. B. Hugh-Jones said that he had derived a great deal of pleasure from reading such a very practical Paper, presented by three leading County Engineers. The combined heater and planer was a piece of plant of which many more were needed in Great Britain. It was a most economical system of improving the riding qualities, and he thought that it ought to be used much more than it had been so far. Now that there were so many different types of planer available, he hoped that it would be possible to do that.

He agreed with Dr Glanville in thinking that it was not always necessary to cover the surface within a few months. There were a good many cases where the road had been left for more than 18 months without ill effects. Each case ought to be considered on its merits.

The economy and the technical excellence of the finished surface laid by a Barber-Greene machine naturally depended on the machine to a ver large extent, but the training of the crew and keeping a good team togethe were of primary importance. Not only should the crew be keen and intelligent but there should be at least one person in the team, and perhaps mor than one, who was familiar with running adjustments and minor repairs The Ministry of Transport owned 13 machines, 12 large and 1 Junior, and in 1951 227,000 tons of material had been laid. The cost of depot repair and major repairs on the site worked out at 7*d*. per ton. The men at th depot were very skilled, so that that was probably a fairly low figure it included a percentage for overhead charges. Having regard to th mileage of roads which ought to be surfaced, there ought to be far mor machines in operation in Great Britain. They were costly, but they paid for themselves very quickly, and he hoped that highway authorities would be more forthcoming in purchasing these machines.

It was sometimes said that the highway authorities could not do any more work because they were short of labour. The labour force on the roads to-day was 35 per cent less than in 1939. It might perhaps be assumed that mechanization would make up for that, but in point of fact it did not seem to do so, and one reason might be that there were not yet enough machines operating.

He could confirm what Mr Richards had said about the tonnages which were laid in America; he had also seen several miles of surfacing being laid in Canada, and there the tonnages were at least equal to those mentioned by Mr Richards, and the quality of the work and of the material had been first-rate. He had ridden over several miles of that road, and the surface was as good as any which he had seen produced in Great Britain, while the price of it was astounding, being 3.60 dollars, supplied and laid, which worked out at 26 shillings per ton. When it was remembered that their minimum rate of pay was 5 shillings per hour, one wondered how they did it; but he had seen it being done and he had also inspected their production arrangements.

He would emphasize what the Authors said about the importance of cleaning the machines, because very great difficulty had been experienced, especially in cold weather, from machines which had been started up in the morning with cold asphalt (from the previous day) stuck to the pan or chains, and the chains would not take it. That led to delay and gave rise to a certain amount of trouble in obtaining spare parts. The manufacturers could help by marking those parts of the machine which were most liable to break with an identification number. The Ministry were trying to do that with their own few machines to overcome the difficulty.

Could the Authors provide any information on the economics of a petrol-driven machine compared with a diesel-engined machine? The Ministry's machines had petrol engines (Ford tractor engines), and petrol was very expensive, whereas diesel fuel was cheap. On the other hand, it was difficult to find mechanics who were expert in maintaining a diesel machine, but there was a good supply of mechanics experienced in the maintenance of a Fordson tractor.

The Authors had stated that the Junior model could lay to 8 feet, but the manufacturers gave the figure as 8 feet 6 inches.

Mr Stanley Mehew said that he was sorry that the Authors, despite their terms of reference given in the first paragraph of the Introduction, had been unable to give a little comparative information on another mechanized type of surfacing construction, namely the process now commonly called the "retread" process. It was entirely mechanized, and of particular value because one of the bottlenecks in the use of the Barber-Greene in many parts of Great Britain was the available output of coated material. For rural roads the retread process provided a substitute method which he believed had met with considerable success in certain areas.

With regard to heating and planing, could the Authors give more detailed information on the problem of removing considerable thicknesses, such as a complete old carpet up to about 1½ inch thick? The subject was mentioned in the Paper, but he would be glad to know whether that could be done satisfactorily in cases where it was not merely a matter of dealing with a considerable thickness of accumulated surface dressing. That was important, because in many places carpets had to be replaced and it was not desired to raise the level of the surface because of interference with kerbs and so on.

He was of opinion that the danger of reducing adhesion to the subsequent surfacing was quite a serious one, particularly on roads of some gradient in areas where exceptionally heavy loads were carried. He had experienced a number of failures apparently caused by the surfacing not gripping the old base, and he had assumed the reason to be that the heating-and-planing process had left the old surface very dry. A tack coat could be used, and at times was used, but any comments which the Authors had to make on the problem would be of interest. In that connexion, he would like to know whether the Authors favoured the surface dressing, possibly with slag shippings, of a planed surface and allowing traffic on it for a year or two, rather than laying the resurfacing carpet straight away. Mr Mehew favoured that treatment where time permitted.

Referring to the effect of traffic in lessening minor irregularities in surfaces laid with Barber-Greene machines, he had found it of considerable advantage, when laying two-coat work to allow the traffic to run on the sealed base coat for at least a year, and then follow up with the finishing coat. He had carried out experiments, and he was convinced that a longer-lasting and better surface was obtained by leaving an interval between the laying of the base coat and the final coat. Would the Authors give their views on that?

With regard to economics, the theoretical approach mentioned in the latter part of the Paper was of very great interest. The Authors had calculated that at the standard speed a machine should be capable of laying about 1,100 tons a day. That was a very interesting approach, which emphasized the large difference between the actual and the theoretical performance. On the allied subject of mechanized surface dressing, for instance, he had recently investigated the question on the same lines with rather staggering results, because he found that if a mechanical pressure sprayer put down 2,500 gallons of binder a day, which was quite a good average figure, the machine on straightforward running would be operating for only about 47 minutes.

On the question of evenly planned deliveries in order to prevent stoppages, referred to on p. 677, the Authors would appreciate that highway authorities were very much in the hands of the suppliers of materials, because very few highway authorities manufactured all their own materials. Mr Mehew said that in his own area in Derbyshire, in which there were many producers of coated stone, some difficulty had been experienced in getting good deliveries in the mornings, for the simple reason that the manufacturers wanted to deliver to their more distant customers first.

He complimented the Authors on *Fig. 1*, which was most ingenious and instructive. It was interesting to see that on those two jobs, which he gathered were about 10 to 15 miles from the plant, most of the lorries made only two or three trips a day, despite the comparatively short haul. It would appear that the delays could be reduced with advantage to efficiency. The many difficulties in doing that were appreciated, and he merely wished to emphasize how valuable the diagram was in illustrating where the delays occurred.

Regarding big deliveries, did the Authors feel that the human element was of great importance in handling them? He felt that there was a tendency, which he had noticed also with surface dressing, for the men in the gang to "fade" a little if very large quantities were supplied to the job. In saying that he had in mind figures well in excess of 200 tons per day. Could the Authors give any information on United States or Canadian practice in that matter? Were the men dealing with those large tonnages referred to by the Authors paid on an incentive scheme or output-bonus system?

Mr H. S. Andrew said that one of the great attractions of the Paper was that it dealt fully with a comparatively small but not unimportant aspect of the subject, and it gained very greatly in interest as a result of that.

The Authors had shown that the quality of hand-laying was surpassed by that of machine-laying. The Authors also made a good point when they emphasized that with the Barber-Greene machine, as distinct from other methods, differential compaction did not occur under the roller. That was the crux of the matter when laying on an imperfect base coat, as had often to be done.

From Table 2, taking the average figure of 1,100 tons per day, which could be reasonably achieved on an actual productive time varying between

11 and 33 per cent of the available working time, it would appear that the Americans achieved an output of between 121 and 363 tons per day, which figures, he thought, were surpassed by the average figures in Great Britain in many cases. It was striking to note that on an average only 21 per cent of the available working time was used in America.

He regarded pp. 680 to 691 as probably the most valuable part of the Paper. The economic use of the Barber-Greene demanded a full knowledge of its abilities and capabilities, and they were very well given by the Authors.

The figures for the pay and output of American operators would be noted with interest. All those who were connected with the roads in Great Britain were anxious to see the pay of the British operator increased, and to see him given machines which would increase his efficiency in order that his pay could be increased, perhaps not to American levels but to a level which would make it possible to keep a reasonable amount of good labour on the roads of the country.

Possibly the second paragraph on p. 683 should be amplified, because it was not clear what accumulation of material should be on the job before the machine started.

Mr Andrew agreed with the statement on p. 689 that "It is usually better to mark service covers and lay over them and to re-set after surfacing has been completed," but said that on occasion the reverse had appeared to be more suitable and had some advantages, if only in the view of people who liked to think that the job was finished when the surfacing was seen going down. Moreover, it had some merit in the days when there was apt to be a shortage of the type of labour required for the purpose. One of the most interesting parts of the Paper was on p. 669, where the Authors referred to the economic implications of the change in conditions. The temptation afforded by the method of putting down a carpet which could be burned off and replaced cheaply and readily, and with very little interference with traffic, must have a very great effect on the whole outlook on the construction of roads, at least in rural areas. There would be a tendency to consider the treatment of a long length of road in one piece during one year, as compared with the pre-war method of dealing with very much shorter lengths of the same roads.

He tended to have an urban outlook on these matters, and there was another aspect of the subject which he thought required more careful consideration than had been given to it in the past. Manholes and gulley grates between the kerbs of carriageways must be abolished. That was a point which should have careful consideration both in the New Towns and, more particularly, on the Motorways. His own authority had at times in the past been guilty of putting manholes on their surface-water drains in the carriageways. That should be avoided in the future, and if an inset gulley grate which was satisfactory could be found, they might be able to dispense with the mason's labour following on the process o planing and carpeting.

Mr A. J. Lyddon expressed alarm lest there be a future Chancello of the Exchequer present who had listened to the remarks of Mr Richard and Mr Hugh-Jones. Mr Richards had referred to what had been done i 1947 in the way of resurfacing 44 miles of road, and had said that at n greater cost that could now be doubled or trebled. Mr Hugh-Jones sai "Take a skim off and let it go on for some years." Extreme care ought t be taken not to do or say anything which would make it more difficult t persuade the authorities to provide more money.

On p. 9 the Authors had said that "the technique of specifying an manufacturing surfacing materials has advanced to-day to a stage of almost complete reliability." There would be general agreement with that. O even greater importance, perhaps, was the statement on the next pag that "great care should be taken in the provision of adequate drainag for the base on which the surfacing material is laid." That should be trebl underlined, because many of the defects of the materials laid arose from want of drainage. He admitted that the money made available had not enabled as much to be spent on drainage as could be wished. The Authors were very wise in emphasizing the importance of the provision of adequate drainage.

With the knowledge possessed by the Authors from their experience in three important counties, Lanarkshire, the West Riding, and Hampshire, he had been hoping to find in the Paper comparable figures for English practice as well as figures for American practice. He had no wish to detract from what was being done in America, but he was not disposed to accept a position of inferiority for Great Britain on the question of output. It might be true that figures such as those quoted were not obtained on British highways, but he had known one single Barber-Greene machine to lay 1,000 to 1,200 tons day after day, not on highways but on runways. It must not be said, therefore, that it was not possible to do it. If the users of the material would co-ordinate their demands or programme their requirements it would be quite possible to double or treble the quantity of material laid by a single machine. The difficulties were not engineering difficulties, of course, and the position was conditioned entirely by the need to make the money available go as far as possible, so that it might be possible to lay quantities, in this country, which were quoted by the Authors as being laid in America.

On p. 681 the Authors had said that "To maintain minimum thickness and a level surface should be the aim." That again should be underlined. They went on to remark that "The material used should be measured and paid for by the ton, so that there will be no inducement to depart from this principle," which had often been said. A good riding surface must be and could be obtained, but if the sub-base were irregular it must be regulated and dealt with as the Authors had suggested. He had been told that in Hampshire, before the war, £300,000 a year had been spent on preparatory work before surfacing. That had been eliminated now, but the thicknesses of surfacing material had been increased, as was corroborated by the figures given by Mr Richards. There were two schools of thought on the matter, and there were many who said "Do not trouble about the minor irregularities, but put down a reasonable coating so that the Barber-Greene can almost eliminate the irregularities by reason of the adequate thickness laid."

It must not be forgotten that we were not behindhand in productive capacity in Great Britain. He could name three or four plants which turned out 10,000 tons per week. The material had to be distributed to a very large number of users, but our productive methods were not far behind the best. In certain parts of the U.S.A. the whole of the productive capacity was conditioned by getting the work done in the minimum of time for climatic reasons. There were not many countries in the world where it was possible for work to be carried on for 300 days in the year (as in Great Britain) and that made a difference to the output over the year.

Another point to be borne in mind was that 5 or 6 years ago the then Minister of Transport had referred to a 10-year programme of road construction and maintenance. Acting on that, one firm at least spent more than £1,000,000 on modernizing and enlarging their plant—a great risk to take in view of the subsequent paucity of funds.

Mr H. W. Hayton thanked Mr Lovell for making some correction of the figures given in the Paper for the cost of planing. From recent information Mr Hayton had found that costs were now about two-thirds of those given in Table 1.

He then described a development which had not been mentioned so far. He said that a number of operators had incorporated a pick-up elevator to the machines and a device for reclaiming and re-mixing, adding back some of the volatile constituents which had been "burnt out." The product had formerly been rather harsh, but nowadays a material satisfactory for footpaths and repairing the frayed edges of carriageways was obtainable, and the cost was satisfactory at about 1½*d.* extra per yard of planing.

Mr Hayton went on to describe some experiments carried out on sections of the same road in West Sussex in collaboration with the Road Research Laboratory. First, the Laboratory's profiling machine was used, and recorded irregularities of 150 to 200 inches. Then three different planers—a fixed-bar type, a reciprocating-bar type, and a rotating-wheel machine—were tried, and in each case there was a reduction of one-third to one half of the first irregularity. The conclusion was reached that the difference was not one of degree, but merely a matter of recording the different surface textures resulting from the passage of each machine, the profiling machine being very sensitive and recording as small a variation as 0·01 inch.

A thin carpet, not more than ⅝ inch, was then laid by a Barber-Greene and the profile machine used again to record any improvement. On each section this reduced the irregularity, from the new datum, by two-thirds to three-quarters, and the final irregularity indexes obtained were between 31 and 35 inches per mile. Those figures were as low as, or slighty lower than, the very best shown on Dr Glanville's slides, and gave an indication of the immense value of those two processes together to the road engineer.

On the subject of the Barber-Greene machine, there were two conflicting statements in the Paper. On p. 687, the Authors had said that it was undesirable to lay short lengths and then change over purposely to match the longitudinal joint, and on the next page they said that traffic would break down the exposed edge, which of course it did. It was difficult to correlate those two statements, because the machine would lay easily a long length of road and it was almost impossible, on a winding road in particular, to prevent traffic running over and damaging the edge. A compromise which he had introduced was to work the machine for the first half of the morning on one side of the road, and then drop back to the other side for the remainder of the morning and half the afternoon, coming back in the afternoon to match up. That seemed to give the smallest amount of exposed joint and the least amount of potential danger to the joint.

Had the Authors any views on the long roll, which was a difficult subject on which to get information? A string line could indicate varying irregularities over lengths of, say, 40, 80, and 120 feet, and if they were all ironed out, the result would be almost a railroad, at colossal expense. From the point of view of comfortable riding, was there any particular length of depression which might safely be left to the ameliorating effect of the machine itself?

Regarding the preliminary regulation—so costly, and using that precious commodity, man-labour—had the Authors experimented with the use of the Barber-Greene, cutting almost bare to lay the first regulating coat on the top of the old road?

Mr J. McCarthy Lee referred to the qualitative tests mentioned on p. 679 and said that, in spite of what had been said earlier, he thought that they were very necessary. In East Sussex different types of material had been laid by the Barber-Greene machine and the conclusion had been reached that whereas materials up to about ¾-inch gauge could be laid quite satisfactorily, above that size there were difficulties, and it seemed very difficult to produce a really perfect surface in one layer with, say, a top course material of 1½-inch-down-gauge tar macadam or with 2½-inch-down mixed-gauge tar macadam. Whether that was the result of their own ignorance in the use of the machine or the ignorance of the suppliers of the tar macadam regarding the material, he did not know. It might be caused by some inherent defect in the machine—it had been suggested that segregation was caused by the action of the feeding screws; could the

Authors give more information on that? It would also be interesting if the Road Research Laboratory could undertake some tests which would produce definite specifications for the larger gauges of material for use by the Barber-Greene machine.

Mr Lee referred to the question of working the machine below capacity. There were no supplies of stone within East Sussex; the two main suppliers were in Kent and West Sussex. The minimum haul from one was more than 8 miles, and from the other more than 12 miles, so that the haul to a site in the middle of the county was at least 20 miles. Each of those firms manufactured different types of tar macadam, and since it would be unwise to mix the two together, attempt was made to get as much as possible from one or other of the suppliers. The result had been that the maximum ever obtained was just more than 200 tons in a day, and that was all that could be obtained unless the different types of material were combined, or more distant sources used. Could the Authors suggest a way out of that difficulty? It seemed to him that the solution was for the suppliers to improve their output. Perhaps the right way to look at it was that the Barber-Greene machine produced a better running surface than it was possible to obtain by hand, and did so more cheaply.

⁂ **Mr H. E. Aldington** said that the comparisons made by the Authors between the results obtained by the use of mechanical plant in the U.S.A. and in Great Britain could be misleading, unless the quite differing circumstances in each country were taken into account. The Authors did not give sufficient weight to that, and consequently appeared to have reached the erroneous conclusion that British methods, mechanical plant design, and their efficiency lagged behind practice in the U.S.A. Comparison between British and American rates of work could be illuminating only if that comparison were faithful, that is to say, direct comparison of site conditions, scale of operations, over-all requirements of working and laying, initial moisture content of aggregates, and ambient temperatures.

It was misleading to compare a nominal 10-tons-per-hour plant with a 160-tons-per-hour plant of any make, because those plants were designed to satisfy entirely different requirements. If a demand existed in Great Britain for 160-tons-per-hour plants for operation by 5 men they could be supplied by British manufacturers. The 160-tons-per-hour plant referred to in the Paper was probably a continuous mixer used on a large constructional work where there was a continuous demand for material over a long period.

American working conditions normally favoured high output, since large constructional works were undertaken on a seasonal basis and not all-the-year-round as in Great Britain. Thus the average moisture content in British aggregates was higher, and ambient temperatures lower, and

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⁂ This contribution was submitted in writing.—Sec. I.C.E.

therefore British stone dryers were rated more conservatively than thei American counterparts. Fuel economy, and consequently thermal efficiency, was also of much greater importance in Great Britain.

The British surfacing-materials plant mentioned appeared to be a credi to British design if it produced 25 tons per hour against a nominal rating of 10 tons per hour. Few, if any, American plants would be capable o exceeding rated output by 150 per cent even in America and when used in this country it would be unlikely that their rated output could be maintained if adherence to mix specifications were observed.

The mere reference to a labour force of 11 to 13 men to operate the British plant was incomprehensible without some clarification. Such a plant normally required 4 to 5 men for actual operation at 25 tons per hour.

The British plant producing 25 tons per hour through a 1-ton mixer allowed 2½ minutes batch cycle or a maximum mixing time of about 2 minutes per batch, which is ample. The American plant producing 160 tons per hour through a 2½-ton mixer allowed a batch cycle of 56 seconds and a maximum possible mixing time of about 30 seconds which, by normal standards for asphalt, was inadequate. American asphalt mixers were certainly not *more* efficient than British mixers, and the remarkable variation between the two mixing cycles could only indicate that British mix specifications placed quality before quantity. 2½-ton mixers were now made in Great Britain and were rated at 60/70 tons per hour: that rating may be conservative, but it would seem quite impossible for any single 2½-ton batch mixer to produce 160 tons per hour as an average rate of working to a controlled specification, especially since the batch cycles quoted above were maximum and took no account of inevitable plant or transport delays.

There were 60/70-tons-per-hour batch-mixing plants in operation in Great Britain delivering material at the rate of 100 to 110 tons per man-day and working to a controlled specification.

The Authors also compared the output per day from the Barber-Greene surfacing machine in the U.S.A. and in Great Britain to our disadvantage. The reason for that difference surely was that owing to the present financial stringency, road maintenance work had to be carried out in "penny numbers" and there was no call for 1,000 tons of material to be laid in any one day, quite apart from the practical difficulty of dealing with that volume of material on narrow winding roads, carrying continuous streams of traffic. In the U.S.A. conditions were quite different. Great Britain had more vehicles per mile of road than any other country.

It should be noted, however, that where there was adequate space, no traffic difficulties, and the money was available to cover the cost of the work as in the case of the resurfacing of runways, it was not unusual for 1,000 tons to be laid per day by means of a Barber-Greene surfacing machine in Great Britain.

The Authors, in reply, said that Dr Glanville had asked whether surfaces which had been heated and planed required to be resurfaced within a short period of being so treated. It was considered that each specific case required to be dealt with on its merits. Probably the two most important factors were :—

(1) That the waterproof seal to the road had not been destroyed.
(2) Any seal remaining was not oxidized by action of heat.

Surfaces which were originally of a "fatty" nature could possibly be retained for a longer period before resurfacing. The results of practical experience indicated that the effects of fairly severe weather conditions which prevailed during the winter period, and particularly wet conditions which extended over several months, necessitated the execution of resurfacing within a few months of heating and planing treatment, otherwise delays were likely to produce deterioration in the surfacing and base.

That tended to show that the possibility of leaving planed surfaces without further treatment was limited to the type of surface provided by dense asphalt more than 1½ inch thick. Another feature which had to be watched was that a planed surface, being quite smooth, often tended to be slippery.

It was necessary to use the term "academic" research because that was what research had come to mean in Great Britain—the investigation of scientific matters by scientists—and any other type of investigation could be defined only by contrast. Reference to the dictionary would show that the term "research" was of far wider connotation and included production and cost studies such as the Paper suggested. Research of that kind aimed at eliciting the magnitude, nature, and cause of delays or other impediments in the productive processes, or by studying how they might be facilitated, and by eliminating the causes of delay and establishing a proper integration between the parts of a process to attain the full potential output of which the process was capable. The attention which those matters received in Britain, outside the factory, in which they were often well established, could only be described as casual, whereas their importance in increasing productivity and reducing cost was fundamental. It could reside both in comparing the comparative value of different processes or in examining the efficiency of a single process and its constituent factors. American output and efficiency were greater than British not because the stone was better or the bitumen was different but because the Americans had given that attentive consideration to the economy of operations which the British had neglected.[1]

From the argument used in connexion with the cost of surfacing material it might be thought that good output and high quality were incompatible, but that argument did not appear to be borne out by fact. Everyone would agree with the desirability of maintaining a high standard of quality, thereby extending the life of the surfacing and reducing its annua cost, though it was a little difficult to believe that doubling the life of th surfacing materials being laid today was a practicable matter except i those rare cases which might arise from slipshod manufacture. Th argument of the Paper was that the production of road surfaces was quantitative as well as a qualitative process and that there was room fo quantitative improvement after qualitative excellence had been obtained The surfaces being laid at high output and low cost in the United State and Canada, as confirmed by Mr Hugh Jones, were of excellent qualit and at least equal to those produced in Great Britain. However, ther was no doubt that the millenium with regard to the quality or quantit of surfacing materials had not yet been reached.

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[1] E. W. W. Richards, "Production and Cost Studies in Highway Engineering." *Rds and Rd Constr.*, vol. 30, p. 234 (Aug. 1952).

The question of qualitative tests and desired characteristics and th tests developed in the United States of America was a very large one It had been dealt with it in an article [2] by one of the Authors.

Dr Glanville had also queried the output capacity of the radio equipment at 25 miles range, as mentioned by the Authors in relation to the Post Office recommendations. In that respect the following factor determined the range of the equipment :—

(1) Topography of the area to ascertain how the horizon would clear.
(2) The geology of the area to determine the extent of ground conductivity of intervening terrain.
(3) The combined horizons of both stations in contact. The radio horizon was slightly greater than the optical and was largely determined by the height of the aerials above surrounding obstructions.
(4) The signal/noise ratio of the signal received and the sensitivity of the receivers.
(5) The percentage of modulation of the carrier wave of the transmitter. The power output, although important, was not the most important factor. In V.H.F. it had been found that there was little point in using more than 25 watts.

An approximate formula for range of radio equipment was :—

$$\text{Distance (miles)} = 1{\cdot}225 \times \text{vert. height of aerial No. 1 above MSL.} \\ + 1{\cdot}225 \times \text{vert. height of aerial No. 2 above MSL.}$$

The Post Office estimate of 15 miles radius was based on the use of pre-war equipment and average siting of aerials.

Mr Hugh Jones had appeared to infer that highway authorities, for several reasons, had a limited capacity for carrying out work. That inference could not be generally accepted, and, in fact, he had mentioned one way in which capacity could be increased, namely, mechanization.

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[2] E. W. W. Richards, "Bituminous Surfacing on Roads and Airfields by Mechanical Finisher." *Rds and Rd Constr.*, vol. 30, p. 196 (July, 1952).

There was little doubt that, given the money, highway authorities throughout the country could compete with very considerably extended programmes; for instance, in the West Riding of Yorkshire where mechanization was fairly well advanced, but not complete, the productive part of a £2,250,000 annual expenditure could be executed with comparative ease in the first 6 months of the year, and that rate of progress could be speeded up without any difficulty should the necessity arise.

It was always difficult to make more than a momentary comparison between petrol- and diesel-engined equipment, since as soon as a large enough number of users turned over to the cheaper equipment the Government proceeded to redress the loss of revenue by increasing the tax on the cheaper fuel. The present position was that the diesel-engined machine was about £450 more expensive to buy but that the fuel economy per 8-hour day was of the order of 23*s.* so that the increased capital outlay was eventually recovered.

The figure of 8 feet given in the Paper had been supplied by the manufacturers of the machines, Jack Olding and Company Limited.

The Authors shared with Mr Mehew his regret that both their terms of reference and lack of space prevented dealing with retread processes, particularly in view of his remark that the operation invited a fully mechanized process. It was known that some very considerable success has attended the operation in various parts of the country, the work having been carried out on all classes of road, and not necessarily limited to rural roads. However, as the point had been excluded from the main Paper it would seem inappropriate to quote typical examples, although the Authors were aware of many.

The question of removing considerable thicknesses of surface by heating and planing appeared to vary in degree of success directly with the texture of the material being planed. The finer the material, the easier it was to plane, the coarser materials being inclined to tear. From that it should not be construed that the removal of, say, a thick carpet was an easy operation. For instance, it had been found that the excessive heating required to remove a thick carpet had, in cases where the carpet or asphalt had been laid on a tarmacadam base, affected the base to an extent whereby sections of it were disturbed and lifted during planing. Whilst the planing machines themselves would do the work it was an intricate operation and best attempted in several passes of the planing machine, each with the requisite degree of heating.

It was agreed that the danger of reducing adhesion and subsequent surfacing left by planing was a serious one, because the surface was often both dry and slippery. The use of a tack coat in that connexion was generally sufficient but again the Authors agreed that in places of severe stress such as gradients or where braking or gear-changing occurred regularly, or at bus stops, an additional precaution such as scouring of the old road surfaces was desirable. Further, weather conditions played an important part. Any moisture on the road at the time of laying th new surface acted as a lubricant and encouraged slipping. Again, th finer the texture of the surfacing material generally the greater was th possibility of slipping.

They also agreed entirely that a surface dressing and chippings on planed surface executed a year, or possibly 2 years before the final surfacing, was a desirable feature. However, time was the crucial factor in that case and where the resurfacing had to be done in the same year as th heating and planing the Authors pointed out how those contiguous operations could be made entirely successful.

It was undoubtedly better to leave a time interval between the firs and second courses of two-coat surfacing. The finish obtained was better because the action of the finisher was incremental and successive passes smoothed out irregularities. Since considerable thicknesses may be involved in two-coat surfaces, the base benefited from being allowed to settle down.

Finisher operation was not a manual process and the handling of quantities far larger than were customarily laid in Great Britain should not impose any hardship on the men engaged in the work. All that was required was good organization and an understanding of how the machine operated. The finisher should then be left to get on with the job. Anything required beyond that was a sign that there was something basically wrong, and it should be tackled at the source. American practice in the payment of bonuses was as variable as in Britain but it should not be thought that any *ad hoc* inducement was necessarily required to enable large quantities to be laid. The economic inducement existed elsewhere in the country's pay and taxation system, and the handling of those tonnages was commonplace. The American laying gang ordinarily consisted of three men (including operators) though they sometimes added a fourth to guide trucks on to the finisher when the input was unusually large.

The Authors felt that Mr Andrew had drawn an incorrect inference from Table 2 by considering it in isolation without reference to the supplementary information given in Tables 3 and 4. If the effect of the first two items in Table 3 were excluded, for they were of random and often uncontrollable occurrence, the residual time was increased to 78 per cent of which 22 per cent total available time was consumed in minor delays, leaving a net available working time (average) of 56 per cent. At that rate the average outputs ranged from 550 to 1,600 tons per day, which was a usual daily working range, according to the thicknesses and type of material being laid. It should be emphasized that the material was not being slapped down on airfields in great thicknesses but was being laid to high standards of quality and smoothness.

There should preferably be a sufficient accumulation to ensure continous laying, having regard to plant and finisher rates, haul distances, and number of supply units, etc.

The tendency referred to by Mr Andrew to consider long lengths of road instead of short isolated lengths was perhaps the most efficacious result of mechanization from every point of view. It was sometimes referred to as "routed expenditure," that is to say, dealing with everything necessary on one route in continuous lengths. It undoubtedly increased the safety value of the road, because the road user could expect similar conditions, and was not confronted with unexpected hazards. It was suggested that that approach to highway works was of paramount importance and the most valuable single factor that could be introduced into post-war highway works.

The co-ordination of the users programme for materials was certainly a desirable feature. It did not seem impracticable and was worthy of serious thought and probably of ultimate adoption. There was little doubt that if that was done Mr Lyddon's contention would be entirely proved.

The Authors agreed with the point made by Mr Hayton that an automatic pick-up was now incorporated in many heating and planing machines. That development had been introduced since the Paper was written and was proving very valuable and economical in use. The statements referred to by Mr Hayton and given on p. 687 were not really contradictory. They led up to the conclusion that Mr Hayton himself had come to, of attempting the requisite amount of work on a length of road whereby the full width could be finished during the same day—as Mr Hayton had suggested for two-lane carriageways and suitable modifications in length for carriageways with more than two lanes.

The edges of the laid material standing up under traffic was considerably assisted by the use of a straight-run bitumen. (That point was made as an incidental reply to Mr Hayton, and was not really within the scope of the Paper.)

With reference to Mr Hayton's remarks on a long roll, it was a matter of both length and depth, and of vehicular suspension characteristics. Sway and bump were likewise subjective psychological phenomena. The length of depression which could be corrected by the machine or operator had been defined in the Paper. Some work in the perceptive lengths of depressions had been done at the Road Research Laboratory, but it would probably be a long time before specific guidance could be given.

The Authors felt that the best answer to preliminary irregularities was the use of the heating and planing machine. The Barber-Greene could, of course, be used for filling up major depressions.

With reference to Mr McCarthy Lee's remarks on large-gauge material, one should not expect to produce a "really perfect" surface with very-large-gauge materials, but a very good surface could be produced in 1½-inch and 2¼-inch materials with proper methods, and the action of the feeding screws should be to reduce rather than cause segregation.

It was probable that Mr Lee's difficulties with regard to supplies of materials from two different sources could be overcome by making use of his own specification for materials so that all the manufacturers had the same standard to work to. Certainly, so far as any base-course material was concerned, any slight deviation from the specification would not unduly matter and certainly would not show. With regard to the top courses, however, even the same specification might result in appearances not being uniform, owing to a difference in origin of the aggregate. Generally it would seem that the answer to Mr Lee's question was :—

(1) to produce his own specification ; and
(2) to create the demand for the manufacture of materials in such a quantity so as to be economically worth while for the producers.

In general, the Authors felt that Mr Aldington would agree that there could be no harm done in setting the targets high enough, but, dealing with the specific points he had raised, the comparisons made were faithful so far as could be. The scale of operations was approximately correct, the only difference being that the Americans did in one day what the British accomplished in a week. That was the point of efficiency that the Paper was intended to emphasize, and matters could and should be so organized that the same efficiency could be obtained in Britain.

With reference to the remarks that few American plants would be capable of exceeding rated output by 150 per cent, the point the Authors had been attempting to convey was not a comparison between the types of plant but that plant (and jobs) should be planned for high output and economical operation. The figures given in connexion with the labour force applied to modern plants belonging to local authorities and well-known contractors. Where reference was made to the mixing times of material, the point the Paper had tried to make was that the present mix designs were not necessarily ideal and it might be necessary to design a mix to suit the capacity of the machines; to explore that line of thought was at least desirable from every point of view. There seemed to be no reason why the Americans' superiority should not be acknowledged, otherwise progress in Britain might be stifled. Whilst it was, perhaps, easy to say that because quantity was greater, quality was inevitably worse, the fact was that the quality was better also. Probably the only answer to the disbelieving was to say " Go and see." Mr Hugh Jones had confirmed those points, and in the figures he quoted lay the answer. How did the Americans produce better materials, better laid at those rates for one-third of the cost in Britain ? The men engaged were paid £20 to £40 per week. That was what the Paper had been attempting to show.

With regard to maintenance work being done in penny numbers at present, there were many road jobs of sufficient size in Great Britain which would take materials at the rate of 1,000 tons per day, but had that ever been done, or even attempted ? The argument of the Paper was that it could be done with proper equipment and organization. The traffic conditions really called for more efficiency in organization, and should not be taken as a reason for not meeting the deficiency.

It was agreed that on runways such capacities had been achieved, but it was suggested that if adequate space and traffic difficulties were accepted as a reason why the target should not be achieved on a road, then no progress would be made. After all, what happened to traffic and space during surface dressing operations with a pressure tanker? An equal amount of plant was on the road and the areas covered per day would certainly be equal to that covered by 1,000 tons of resurfacing materials.

Correspondence

Dr A. R. Lee observed that, although the Authors had given some interesting and useful advice on the operation of the Barber-Greene Finisher, no mention had been made of the power-operated blade grader. That type of machine was, as Mr Richards would know, the maid-of-all-work to the American highway engineer. The towed blade grader had been used with great success in Great Britain by one commercial firm in providing first-class level running surfaces, and Dr Lee thought that a logical development would be the use of the power-operated machine for carrying out the process at a faster speed. He understood that Mr Lovell himself was very keen on that machine, and felt that British engineers would like to know if there were reasons why it could not be used more widely in Great Britain.

Dr Lee noted that only a brief reference had been made to the latest American Jaeger finishing machine, which was constructed so that the width of material spread could be varied without interrupting the progress of the machine. That machine would seem to have particular advantages for use in Britain and he would like to know the Authors' opinion of it.

With reference to the section of the Paper dealing with Economics, he thought it was very doubtful if the large outputs quoted would be economic for present-day British conditions of road maintenance, which differed very considerably from American conditions. In Great Britain, only relatively short lengths of surfacing were laid with one type of material and, also, work could proceed throughout the whole year. In many parts of America outside work stopped during the winter. In any case, however, he doubted if the figures quoted were really typical of American road practice. Outputs in America varied probably to a much greater extent than they did in Britain, and the figures quoted were, in his opinion, only obtained with the highest-output plants of the continuous type making very large quantities of one particular mix. The New Jersey Turnpike was the most recent large highway construction job to be undertaken in America, and the rate of progress on that job might be taken as a good example for the upper limit for road construction. Dr Lee's colleague, Mr D. B. Waters, B.Sc., A.M.I.C.E., had visited the work in 1951, and had been given data which showed that on 6 consecutive days, each of 11 hours working time, 1,800 tons (American) of asphalt had been laid per day by two Barber-Greene machines. That was equivalent to 600 tons (English) per day for an 8-hour working day for one machine (not allowing for any breaks for meals). The material had been laid in two courses to a total thickness of 4½ inches and had been delivered to the machines in twenty trucks each holding 20 tons of asphalt.

Although he agreed with the Authors that there was room for improvement in the output of British asphalt and tarmacadam plants, Dr Lee did not think the comparison (made on p. 20) with American plant was a fair one. The British plant, with a 1-ton mixer, would have a normal output of 20 tons per hour of hot asphalt; a plant with a 2-ton mixer would normally have an output of 40 tons per hour. However, he thought it should be possible to increase the output of present British plants without having to make very serious modifications, provided that British engineers were prepared to use more power and more fuel and to use faster mixing speeds. If British and American plant of similar type were compared, it would be found that American mixers were usually run at twice the speed of British mixers, and that American driers, by virtue of great tilting and the use of larger flames and correspondingly greater fuel consumption, dealt with twice the throughput of English driers. With those modifications, therefore, British plants could be made to give the same output as American plants having the same size mixers.

Increased rate of output, however, should be considered in relation to the quality of the material, and Dr Lee's impression was that American surfacings did not give the life of British surfacings. From data issued by the Bureau of Public Roads, the average life of asphaltic concrete in America was about 15 years; that life included the period of normal maintenance, such as surface treatment, until the material was taken out of service as a surfacing. Rolled asphalt in Great Britain had an average life of about 20 years before any maintenance treatment was required.

It was interesting to note the three categories described on p. 680 for the degree of irregularity in the base to be surfaced by the Barber-Greene machine. Dr Lee thought most engineers would themselves think in terms of those categories; the assistance they would expect from the Authors would be a yard-stick by which those categories should be defined. Would the Authors indicate the type of measurements the engineer should make and the limits by which he could define the three categories?

Another point on which Dr Lee had hoped the Authors would have given quantitative data was the limitation on the variation in temperature which could be permitted for satisfactory laying of rolled asphalt. The compositions of bituminous surfacing materials commonly used in Britain were clearly defined by appropriate British Standards, which also gave details of temperature of manufacture. It would be very helpful if the

Authors would indicate the temperature limits for laying the various materials. Dr Lee referred in particular to tarmacadam (B.S. 802), bitumen macadam (B.S. 1621), and rolled asphalt (B.S. 594) with its fourteen schedules.

The latest compositions which had been agreed for the various British Standards for bituminous surfacing materials had been decided primarily in the light of the results of research and experience with materials of high quality and durability, and experience had shown that they were capable of being easily laid with a Barber-Greene Finisher. Dr Lee was not clear as to what further changes the Authors had in mind. It seemed to him to be a wrong principle that the basic design of a surfacing material should be decided by the performance of an existing finishing machine. Also, it was bad economics, as Dr Glanville had shown, for quality and durability to be sacrificed to output. A surfacing should be designed to give the most durable non-skid life possible, although Dr Lee agreed, of course, that the material, at the same time, should be one which could be laid by machine.

Dr Lee thought that an engineer who had had no previous experience with the Barber-Greene machine might be somewhat confused if he tried to decide, with the aid of *Fig. 2*, which of the thirty-three possible causes was the one that had produced a "rough uneven surface."

The Authors, in reply to Dr Lee, remarked that he was perfectly correct in describing the power-operated blade grader as a "maid-of-all-work," and one of the Authors, Mr Lovell, had already two of those machines in operation, with a third on order, and he had found that he could keep those machines more than occupied. However, the reason for the omission of any reference to those machines in the Paper had been covered by the terms of reference set and explained in the opening part of the Paper, and a description of its use would have unfortunately fallen into the same category as the question raised by Mr Mehew on retread operations. The use of the power-operated blade grader could itself form the subject of a separate Paper. The Jager finisher was comparatively new and little experience in its use was available. It worked on the loose strike-off principle and was not always advantageous. It would have seemed a little ambitious to expect the operator to maintain a three-dimensional control, that was, up and down, and in and out. In any case, bleeding-out from the Barber-Greene Finisher rarely presented any real difficulty.

With reference to the economics of daily large outputs, the underlying principle of the Paper in that respect had been dealt with in the Author's reply to the discussion. Whilst work could proceed throughout the year it should be organized so that it did. Surely it was better to complete the work during the fine summer weather. The tonnages quoted had been quite general and included contract and municipal work. About 700 or 800 tons a day was normal and 1,200 tons had been laid. No continuous-mix plants were in use; they were all of batch-type.

Two machines were laying 2,500 American tons in an 8-hour day. The surfacing was being laid at 30 feet per minute, 12 feet wide and 1½ inch thick. That allowed ample time to lay 1,200 tons in an 8-hour day. The size of the truck seemed to be irrelevant so long as the finisher was properly fed.

With reference to the speed of mixers and the tilting of drier-drums, the statement would appear to confirm the Authors' contention that material could be mixed more quickly and cheaply, and the obvious corollary was—why not make use of those modifications?

With reference to the life of materials, many factors entered into the question. The crux of the situation was the cost per year of life. It was not always desirable to have a surface that would last too long, particularly in industrial areas where the surfaces were being continually broken up by Public Utility undertakings. The yard-stick to be used in measuring the economical aspect of the life of material needed very careful thought and might well vary under different conditions.

The Authors felt that the yard-stick to be adopted for measuring the degree of irregularity in the base to be surfaced should be that the engineer should survey the base and make his preparatory plans accordingly, deciding how he could and would deal with each irregularity. That method was described in the Paper.

With reference to temperature limits for laying various materials, to deal adequately with that point would need far more space than was available. However, they felt that the correct approach was to specify a minimum temperature for delivery of materials, which was a normal practice so far as the Authors were aware, and minimum and maximum mixing temperatures. The range at which the material could be laid was automatically the difference between the delivery and the mixing temperatures. Some typical minimum delivery temperatures had been specified for the north of England as follows:—

Tarmacadam	*Not less than*	
Single and base course (32–34° C. E.V.T. tar)	15° C. (60° F.)	
,, ,, ,, (38–40° C. E.V.T. tar)	40° C. (104° F.)	
Bituminous macadam	40° C. (105° F.)	
Bituminous carpet, medium texture	80° C. (170° F.)	(Straight-run bitumen
Hot-rolled asphalt—delivery temperatures between	120° C. (250° F.) and 150° C. (300° F.)	

The Authors agreed that an engineer with no previous experience might have difficulty in deciding which factor caused a bad job when using *Fig. 2*. That only showed what considerable care was necessary to produce a good surface with a Barber-Greene.

The other points made by Dr Lee had been dealt with in the Authors' replies to Mr Aldington's contribution to the oral discussion.

RAILWAY ENGINEERING DIVISION MEETING

13 May, 1952

Mr J. C. L. Train, M.C., M.I.C.E., Chairman of the Division,
in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Railway Paper No. 46

"Factors Involved in the Renewal of Underline and Overline Bridges and their Influence on Design"

by

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SYNOPSIS

Bridge work on the Western Region is carried out by the District Engineer. Superstructures of steel or concrete are designed by the Civil Engineer's Bridge Assistant and abutments by the Assistant Engineer (New Works) or the District Engineer. Renewal of superstructures often results in increase of accommodation and so, in turn, the building of new abutments is called for in timbered trenches spanned by waybeams.

Superstructures are erected, complete or piecemeal, by crane, by rolling-in, or by a combination of these methods, and a design reducing erection to as few crane-lifts as possible is desirable for economy of erection and for minimum occupation of the line. In this respect, railway facilities in cranes and wagons are of great help.

Such work cannot be undertaken without careful and detailed planning beforehand and the erection scheme should be carefully considered before the final drawings are completed. The co-operation and therefore the careful briefing of all Departments concerned is needed before the work can commence.

It is hoped that standardization and extension of the use of pre-stressed concrete superstructures will increase the output of bridge work and help to overtake arrears with the minimum use of steel.

INTRODUCTION

It was the wish of the Railway Engineering Divisional Board that the Paper on this subject should be by a District Engineer and the Author has, therefore, attempted to give below some account of the organization of bridge renewals from the background of the Bristol District of the Western Region.

Shortage of steel and of staff for the design and execution of bridge work, both during and since the Second World War, has resulted in the annual renewal during those periods of a smaller number of bridges than in the years between the two wars. Consequently, the current pool of experience in such work is less than before the recent war and inevitably the work to be undertaken annually will increase as these arrears are overtaken. From both of these view-points it is hoped that this Paper will be of interest and it is well to state at once that large bridges are not here considered.

GENERAL PROCEDURE—WESTERN REGION

Proposals for renewal must arise as a result of examination, and this is carried out by the staff of the District Engineer. On the Western Region each District has a Bridge Inspector, who carries out for the District Engineer the inspection and the repair and renewal work necessary to culverts, bridges, viaducts, tunnels, and retaining walls.

The Bridge Inspector discusses weekly with his Bridge Examiners their written reports whilst the details are fresh in their minds and he visits all sites which appear to need early attention and reports upon all major defects to the District Engineer who then makes his own examination. The first aim is always to avoid renewal. Thus, to give ready examples, arches in which the rings are separating because of an old movement and lack of header courses between successive rings can often be repaired from underneath by the use of a shield of bent rail ribs and laggings, or by hanging laggings, as in tunnel repair work (see *Fig. 1* *), the loose ring being cut out and replaced by new work, block-bonded to the inner rings. Steel structures can also have their life extended by the judicious use of bolted or welded patch plates which may necessitate partial relief of the load by jacking, if the dead load is a considerable proportion of the total load. Usually, before reaching a final decision upon the complete renewal of a brick, masonry, or concrete structure, tell-tales are watched for a year or two to check whether the cause of the defect is still active.

The Author thinks that District Engineers will generally agree that smoke plates are needed for all structures that have steel (or concrete that may develop hair cracks) exposed to engine blast, and that it is a great economy in design to provide an additional initial thickness of steel at those points of a steel structure where corrosion may be the final criterion for reconstruction, as in the bottom flanges and webs of plate girders spanning the tracks.

If the District Engineer considers reconstruction is necessary he lists the structure in his annual proposals, and if a steel or concrete superstructure is involved, the Bridge Assistant is asked for a corrosion survey and the appropriate calculation of present capacity. The proposal is finally considered by the Civil Engineer during his annual Budget Inspection.

Once renewal is decided upon, the Civil Engineer investigates the need for additional accommodation. If the bridge carries or spans a public road, the decision to reconstruct is notified to the appropriate highway authority so that an agreement can be concluded to cover any desirable increase in road width, head-room, or carrying capacity, or any improvement in road alignment. If the bridge is an occupation or accommodation work, the need to re-produce it is checked, as severance of ownership may have occurred. Similarly, the Chief Regional Officer is asked if provision should be made for additional tracks. Thus it is ensured that the new design fulfils the requirements of all users, and the decision to renew the superstructure may result in the provision of a new substructure also, on a new alignment.

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* All illustrations may be found between pp. 734 and 735.

The design of steel and concrete superstructures and trestles is carried out by the Bridge Assistant, based upon surveys made by his survey staff. Substructural work is designed either by the Assistant Engineer (New Works) or by the District Engineer; in either case a site investigation can be had from the Soil Mechanics Section which is under the Assistant Engineer (New Works). At an early stage, the District Engineer makes his comments upon the erection and future maintenance of the proposed structure and later he estimates for the work which he is to carry out. He also prepares the erection scheme.

The first aim is always to carry out the work by Railway Executive staff, and to illustrate the present facilities, the Bridge Inspector of the Bristol District has at present a Sub-Inspector, a Foreman, and an attached New Works Inspector. He has fifty-one tradesmen and sixty mates and labourers. Assistance in a small proportion of bridge and tunnel work is also given by men supplied by the Direct Labour Organization working under District Supervision.

Plant operators are drawn from the Plant Inspector as required and plant such as compressors, concrete mixers, and portable generators with their lighting equipment and small tools are drawn from the District pool. One utility van and one 3-ton lorry are allocated to bridge work and other road transport is hired as required. The District has one 10-ton travelling steam crane for bridge and permanent way work and other crane power is borrowed as required and as available from adjoining Districts and from the Mechanical and Electrical Engineer and the Motive Power Superintendent.

The expenditure in 1950 and 1951 was as shown in Table 1.

TABLE 1.—EXPENDITURE IN 1950 AND 1951

	1950	1951
Bridge renewals	£13,186	£17,505
Tunnel relining and repairs	£13,727	£27,599
Repairs to bridges and culverts	£16,379	£14,993
Painting	£2,217	£2,840
Repairs under block votes	—	£5,646
Day-to-day maintenance	£18,283	£17,185
Totals	£63,792	£85,768

The work carried out included the reconstruction of five bridges in 1950 and five in 1951, and major repairs to nineteen bridges in 1950, and twenty five in 1951.

Excluding retaining walls, the structures to be maintained by this stat comprise forty tunnels (for total lengths, see Table 2), twenty-three tunne shafts, and nearly fifteen hundred bridges (see Table 3).

TABLE 2.—TOTAL LENGTHS OF TUNNELS MAINTAINED

Single-line tunnels	7,366 lineal yds.
Double-line tunnels.	22,456 lineal yds.
Total . . .	29,822 lineal yds.

TABLE 3.—NUMBERS OF BRIDGES MAINTAINED

	Span				Total No.
	5 ft to 10 ft	10 ft to 40 ft	40 ft to 80 ft	More than 80 ft	
Underline					
Metallic, timber, or R.C.*	61	217	33	37	348
Brick or masonry † . .	166	318	23	13	520
					868
Overline					
Metallic, timber, or R.C.*	—	96	45	15	156
Brick or masonry † . .	—	166	98	30	294
					450
Outside the Line					
Metallic, timber, or R.C.*	5	4	1	1	11
Brick or masonry † . .	5	8	3	—	16
					27
Footbridges	—	31	58	15	104
Viaducts (5 spans or more.)					
Metallic, timber, or R.C.*	10, totalling 6,480 ft				
Brick or masonry † . .	34, „ 11,916 ft				

* Wholly or in part. † Wholly.

THE RENEWAL OF ABUTMENTS AND WINGS

For all mass abutments the Author considers that brick-faced concrete gives a pleasing result, is very simple in construction, and is, in consequence, cheaper than shuttered concrete. The Western Region practice is to build four courses of 18-inch work, followed by three courses of 14-inch work and then five courses of 9-inch work, this succession being repeated as the wall rises. An ample key to the concrete backing is thus obtained and this 3-foot 3-inch lift is capable of taking the pressure of the wet concrete behind it.

Where skew quoins occur, delay can arise in obtaining the necessary squint bricks, if the whole work is to be brick-faced, but this delay is readily overcome by building block quoins of concrete, three courses deep, either in pre-cast or mass concrete and these, in appearance, relieve the mass of brickwork and blend with the concrete bedcourses, copings, string courses, and pilaster caps.

The simplest case of abutment renewal is the building of an entirely new wall behind the existing abutment and clear of it.

Before the start of the sub-structural work the least width of trench to give a reasonable working room must be decided and its timbering should be designed. The net width over foundation can be adopted as the minimum, but will involve working overhand by bricklayers, if the facing described above is used. The choice of width may be influenced by the size of crane skip or the width of decauville wagon to be used for the removal of the excavation and the handling of the new materials, and if the increase in span is small, it often pays to extend the trench to the back of the existing wall rather than leave an intervening dumpling.

A method of trench timbering which the Author has found very effective is illustrated in *Figs 2, 3,* and *4*, and is known, at least in the West, as the "piling frame." It is an excellent method where the ground and weather are such that a run may occur from an open face. It avoids the hurried use of temporary frames. By giving a batter to the poling boards it is possible to increase the trench depth to any amount without setting-in for subsequent frames and the poling boards can be driven into the ground ahead of the excavation, so that no loss of ground occurs in cutting the trench. This keeps the load on the timbering to the minimum and also ensures that the settlement of the temporary beams carrying the tracks over the trench will be as small as possible. The length of poling board, and so the vertical spacing of frames, should be related to the anticipated daily lift of brickwork and/or concrete, which in turn is governed by the design of the facing. The frames can then be withdrawn as the work proceeds, so that at most only the back poling boards will be left in position. The walings adjacent to the facework are strutted from "soldiers" on the new work. All the timber, except occasionally the struts, can be cut from serviceable sleepers.

Having settled the trench timbering, the beams to span the trench must be selected. Since the trench work will be carried out under a temporary restriction of speed, which the Author considers should be 5 or 10 miles per hour, dependent upon site conditions, it is reasonable to check the strength of available beams for British Stress Committee A, B, and C loading at 3 revolutions per second, the additional speed thus allowing for any roughness of top that may develop. The beams bear on sleeper sills at each end, and a fair estimate of effective span is "clear span + one-third way inside each set of sills."

The sills themselves should be close-laid, and on a clay bank it is an advantage to have a second layer of timber, cut from sleepers, laid at right angles to the first layer and bedded on an inch or so of dry cement. The pit thus dug for the sills should have good drainage grips because the pounding of traffic in wet weather can produce a rapid breakdown of the clay. If the beams can be carried upon the back of the existing abutment, timbering is reduced and better running obtains. Although over a wide trench the front sill can be placed close to the back of the trench timbering, it is an advantage to keep it 2 or 3 feet farther back.

For quite narrow trenches, clusters of serviceable rails can be used under the sleepers and timber beams are commonly used for small spans, but such timber must nowadays be subject to careful inspection, even if it can be obtained. The Author favours the use of steel beams under cross-sleeper road, with slow folding wedges between the joists and the underside of the sleepers, thus allowing top to be maintained nicely. Small settlements of the sills are taken up by the folding wedges under the sleepers, and when and if these approach their limit of travel the beams themselves are packed up on the sills and the wedges are again adjusted. This temporary structure, the Author suggests, may prove to be capable of design in pre-stressed concrete, in a form to allow a couple of inches of ballast under the sleepers and a somewhat higher temporary restriction of speed.

The constructional depth of the new structure should be considered in choosing the beams, for if the constructional depth of the new work is greater than that of the temporary work, the new bedstones can be set under traffic during the building of the new wall. In limiting cases, an inch or so of additional thickness on the new bearings, coupled perhaps with a slight and temporary increase in rail level when the beams are placed, will allow this early setting of bedstones to be done.

The depth required for the temporary beams can be considerably reduced by securing the running chairs or base-plates to timber waybeams, each carried between twin steel joists; but in this case all settlement must be taken up by packing on the sills, and the top that can be maintained is not as good as with beams and wedges under sleepers.

It is a very great help to the whole of the trench work if, during the occupation of line for laying the temporary beams, sufficient trench excavation is done to permit the fixing of the top two frames of timbering and the first run of poling boards. This is particularly desirable if the sills are hard against the timbering.

Somewhat different problems arise when the new face-line of the abutment runs through the thickness of the existing abutment. Here a thickening behind the old wall is the first stage to be carried out, and after this has been completed the old facework is cut out, with additional pockets for block bonding of the new skin then to be added. The amount of thickening to be given behind the wall is governed not only by the increase in span but more often by the working room required, and the choice between a uniform thickening and the provision of counterforts behind the wall must, in turn, depend upon the thickness and condition of the panel, that would remain between the counterforts. The greatest economy is generally achieved by the construction of counterforts clear of the 4-foot way, since their excavation often permits the use of lacing rails instead of temporary beams and the size required for design purposes normally gives sufficient working room.

All such thickening work should be well bonded to the back of the existing wall and the Author believes that thorough cleaning of the back before concreting, the drawing of occasional stones or bricks on the back, and the use of short lengths of rail or reinforcing steel set in compo in the old work and concreted in the new, are justified, together with a bed-course of reinforced concrete covering old and new work.

If the abutment is being renewed entirely, because of its poor condition as determined by windows cut through the face, another type of problem is encountered. It is then generally expedient to take down the full height of the wall in short lengths, timbering with boards tucked behind and spanning between the undisturbed work. If the old wall is in extremely bad condition, timber soldiers may also be needed, supported by raking struts. Work of this nature calls for temporary trestling under the superstructure.

RENEWAL OF SUPERSTRUCTURES OF UNDERBRIDGES

Broadly, there are four methods of erection from which the final plan of operations can be chosen, and these are considered below :—

(1) *Erection Complete by Crane*

The word " complete " as used here refers to the erection by one lift of the crane or cranes of sufficient of the new superstructure to carry one track. The Author considers that this is the ideal to be aimed at, since the total time for erection is then at the minimum and handling costs are low. It can only be used for small spans which can be reached from one abutment or where multiple tracks are carried on basically independent structures.

Where shallow constructional depth requires the use of longitudinal steel troughing, it can be lifted-in complete with trough filling and waterproofing, and simple spans of concrete slabs or steel joists in concrete can be similarly handled within the limits of the load gauge and practicabl encroachments upon it.

Simple steel deck spans, up to about 45 feet span, lend themselves wel to the method and the designer does well to consider how such a structur will sit upon a "special wagon" for despatch accordingly to site. Elaborate falsework or cradling should be avoided, since the permanent bracing between the girders should be ample to permit such a bridge to be lifte by a pair of "brothers" or twin-legged slings attached directly to lifting cleats bolted to the top flange of the girders. For comfort, the tension in such bolts should be lifted to 4 tons per square inch on the area at the bottom of the thread and the erectors must be supervised when using spanners to "hand tightness" only on these bolts. The outstanding web of the lifting bracket should be set to the approximate line of the sling, say 60 degrees.

The Author used this method at Cardiff a few years before the war for a through bridge. Queen Street, the main road between Cardiff and Newport, had two pavements and a carriageway with two lines of tramways and was spanned by a series of cast-iron arch ribs with cast-iron lattice spandrils and timber decking. The tramway transmission lines were attached to the underside of the structure which carried one track and part of a single connexion. A plate-girder span ran alongside this cast-iron bridge.

The cast-iron portion was to be replaced by a through span with cross-girders, rail bearers, and plated floor, with the main girders non-parallel in plan. It was erected on timberwork at high level, at a height to allow riveting of the floor. Two lifting brackets, about 5 feet apart, were attached to each of the end cross-girders. A rather tedious but careful calculation was made of the position of the centre of gravity on two axes and the proper position for the lifting brackets and the load to be carried by each crane were determined. The bridge was carried by two 36-ton breakdown cranes and travelled satisfactorily with no falsework: the main girder connexion to the cross-girder, under a cranked main-girder stiffener, gave sufficient fixity for safety, and the reversed stress and deflexion in the end cross-girders was not excessive.

Possession of the roadway was not possible until after the passing of the last tram at about midnight and by 7.0 a.m. the new bridge was on its bearings with only the end bays of plating, over the footpaths, to complete. Each of the cast-iron ribs was lifted out complete with its spandril and dumped alongside for burning into wagon size. Interference with road traffic and cost to the railway undertaking were thus kept to the minimum.

From the erection viewpoint it is to be hoped that pre-stressed concrete spans that permit of such easy handling will eventually develop. Their very modest ratio of depth to span (about one twentieth) will help in reduction of weight, and lifting stresses and the provision of lifting hooks are not a difficult problem.

(2) *Erection Piecemeal by Crane*

This method is naturally slower than (1) but is capable of very general application. As in (1) the greatest economy will be obtained if the steelwork, pre-cast concrete, bedstones, etc., are sent direct from the maker's yards or the Executive's depots to the site, loaded in a pre-determined order so that the desired sequence is followed automatically in unloading. Before the war this was general practice but the shortage of steel and delay in its fabrication, together with the present heavy programming of output from pre-cast concrete depots, make such co-ordination of supply difficult and, especially with heavy concrete work, a considerable expense can be added to the method by engine and crane power if the material has to be off-loaded at site, or even more so, in a District Depot.

It should be remembered that the piecemeal method involves leaving possibly quite big girders standing alone on the abutments whilst other lifts are made over them. They should, therefore, always be secured to avoid the unfortunate results of an accidental hit. The simplest way of doing this is often to lash through a rivet hole to an adjoining girder, using scaffold lashings, or to fix two timber rakers at each end, bearing into the top main angle and against a footblock, which can often be a short rail set in the abutment beforehand for that purpose.

Ample interest has been taken for many years in the detailing of bridgework to permit the easy swinging into position of floor members and in the provision of room to make connexions, but the Author thinks that every effort should now be made to reduce the number of pieces. For speedy erection, under occupation of line, it is fair to say that anything heavier than one cwt ought to be lifted by crane. This means that only one, or at most two, such pieces can be handled at once. The Author's department has recently erected a through-span underbridge (span 40 feet centre to centre of bearings) with pre-cast jack arches between pre-cast skew-backs on steel cross-girders. Each bay between cross-girders entailed four lifts by the crane for the jack arches; yet the final floor should be stiff enough to allow the use of a full length jack arch, needing only one lift. If it is feared that cross-girder deflexion would crack a long jack arch, its depth is such that a quite light reinforcing bar near the springing should remove the grounds for anxiety. Although this particular bridge "came in" very nicely, there were 336 separate pieces to erect or fix in the double line, excluding parapets. Delivery to site required twenty-two wagons, including four special shock-absorbing wagons, whilst the old superstructure left the site in two wagons. (See *Figs 5* and *6*.)

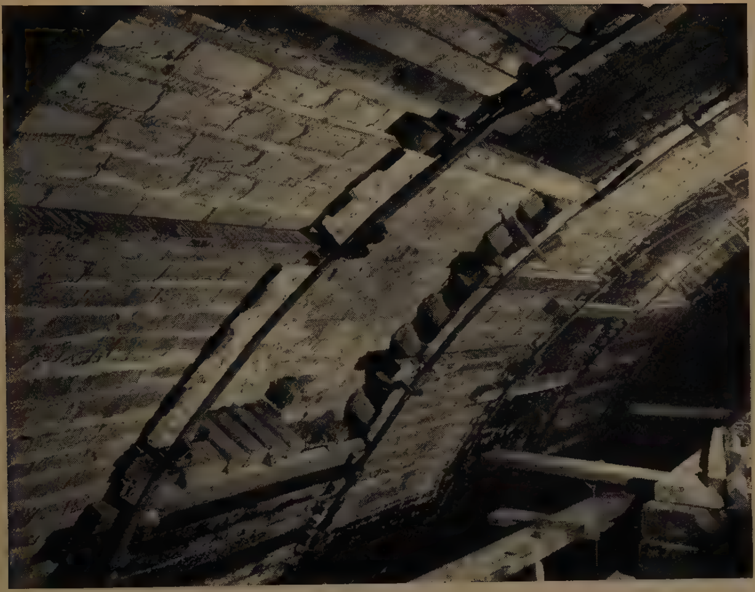
The bigger spans demanding lattice girders present an interesting problem for erection. The top and bottom booms and the web members arrive at site separately and the girders must be built with considerable care to ensure fair holes and line and to give them their proper unladen camber. This building is often done on a falsework over the final span, but if the complete girder is within the capacity of two breakdown cranes propped on their extension girders, it is preferable to build on an adjoining site within reach of the cranes and to take the girders to their final site on Pollen wagons, as an out-of-gauge load, for lifting into position by crane. Not only does this facilitate accurate erection, but a good deal of expense in falsework is saved.

The North Acton–Ruislip line, the major structural portion of which was completed at the outbreak of the 1939–45 war, crossed the West Ealing loop at a considerable skew, calling for two steel bridges, known as "Bridge 'A'" and "Bridge 'B'." (See *Figs 7–11.*) Bridge "A" was a single span, its longest girder being 151 feet 3 inches long between centres of bearings and 12 feet 1 inch deep, and weighing 68 tons; bridge "B" had two spans, the girders being 135 feet 6½ inches long between centres of bearings and 14 feet 6 inches deep, and weighing 62½ tons each. These girders were built alongside a siding about ¼ mile from their final site and were lifted by crane on to Pollen wagons. (Despite their height, they were secured only by chains at each end to short timber baulks across the wagons.) They were then taken over track having 2 inches of cant and through a crossover road. Pending the erection of sufficient cross-girders and outriggers to ensure their stability, the single girders, after landing, were held by 12-inch-by-12-inch baulks from their top booms to posts bolted to the abutment, fixed by a contractor's light crane or from one bridge to the other. For the final landing of one end of a main girder of Bridge "B," this end was lowered on to greased skids and just kept on the crane while the other end was fully carried by the second crane. In that way the girder was effectively steadied and its weight partly taken by a crane through a distance a few feet outside the lifting radius for a clear lift. The four main girders of Bridge "B," with sufficient floor to stabilize them, were erected in 8 hours.

The nearby bridge over the Greenford Canal had main girders which were 147 feet long between centres of bearings and weighed 128 tons each. They were, therefore, beyond the capacity of available crane power and had to be built in situ on temporary bridging involving piling. (See *Fig. 12.*)

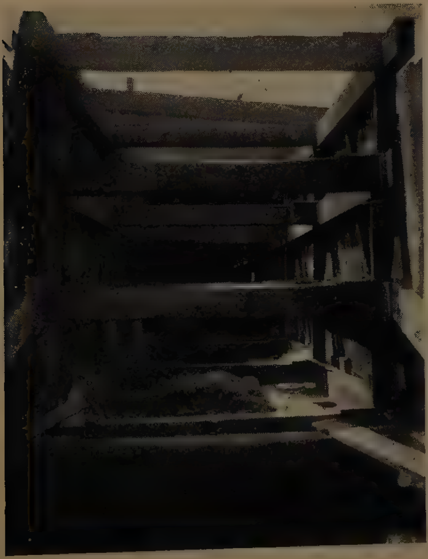
The choice of crane power for piecemeal erection of bridgework is worth some thought. Heavy main girders demand the 45-ton or 36-ton breakdown cranes, whose maximum lifts are slow. An improvement in the speed of setting the extension girders and their jacks is desirable on future cranes, since the "dressing" of a crane may eat seriously into a "between trains" margin, but the Author does not think that any great increase in their rate of lifting or derricking is desirable. If, therefore, the floor steel or concrete units are numerous and the occupation time available leaves little in hand, it may well be worth while to send away the bigger crane or cranes and bring in other crane power of capacity which accords more closely with the lifts and having a greater working speed.

Fig. 1



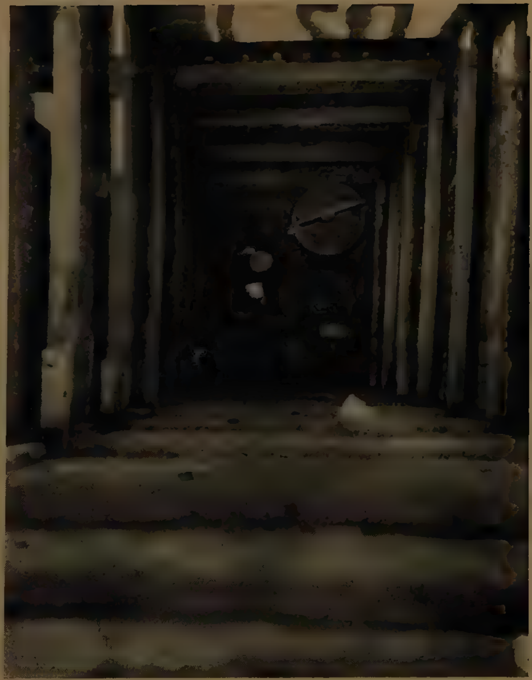
HANGING LAGGINGS IN USE IN SEVERN TUNNEL

Fig. 2



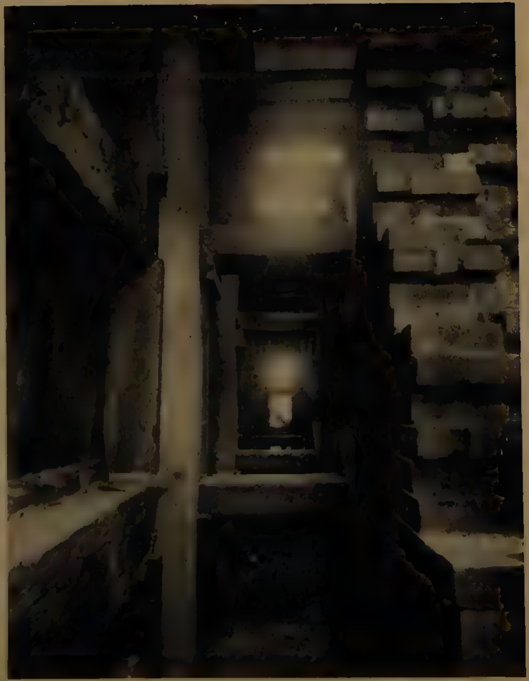
SECTION OF TRENCH USING PILING-FRAME TIMBERING

Fig. 3



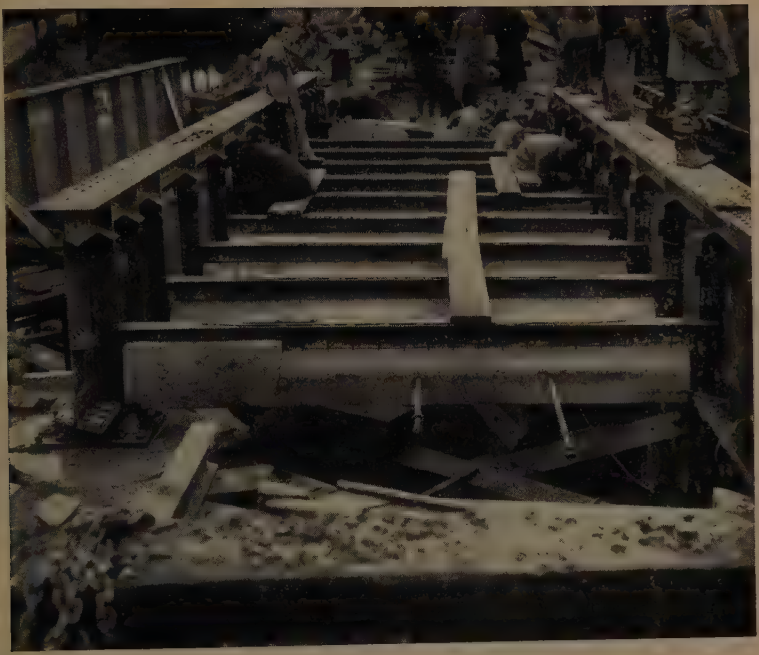
Plan of Piling-Frame Timbering

Fig. 4



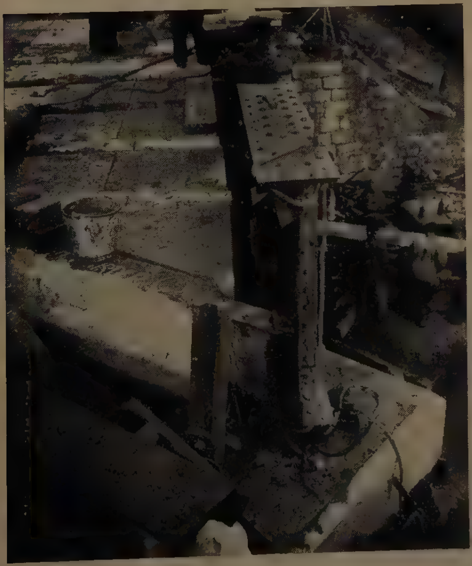
Piling-Frame Timbering as New Work Proceeds

Fig. 5



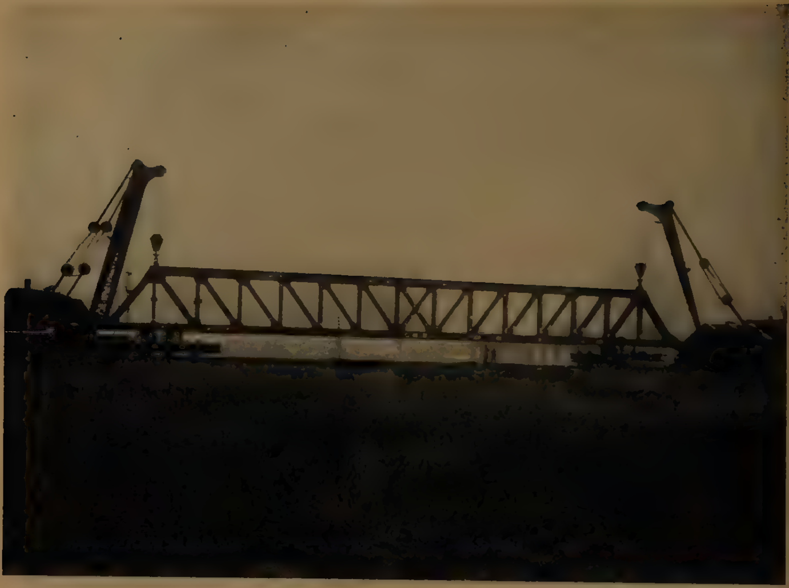
Penleigh Road Bridge, Westbury

Fig. 6



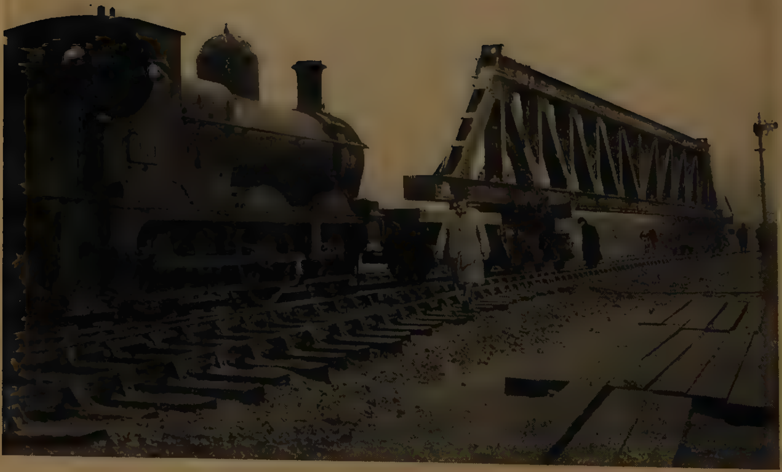
Pre-cast Jack-Arches, Penleigh Road Bridge, Westbury

Fig. 7



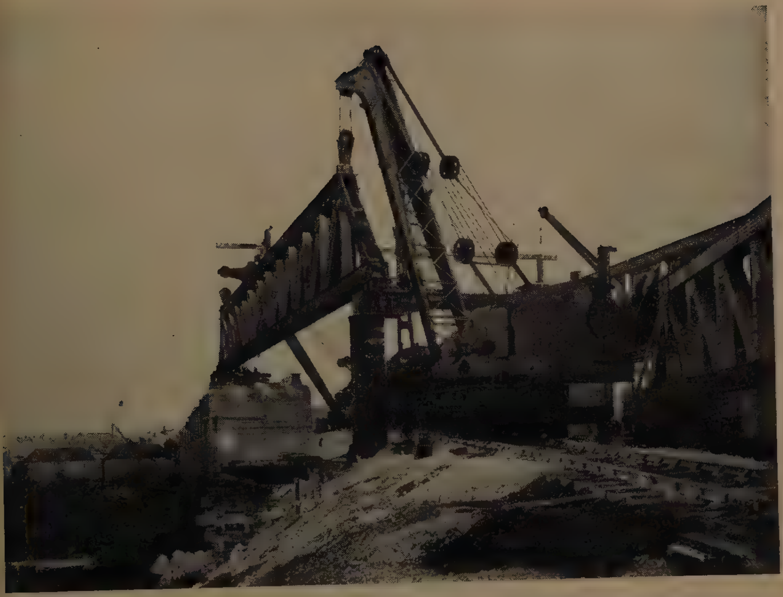
Landing Main Girder on Pollen Wagons

Fig. 8



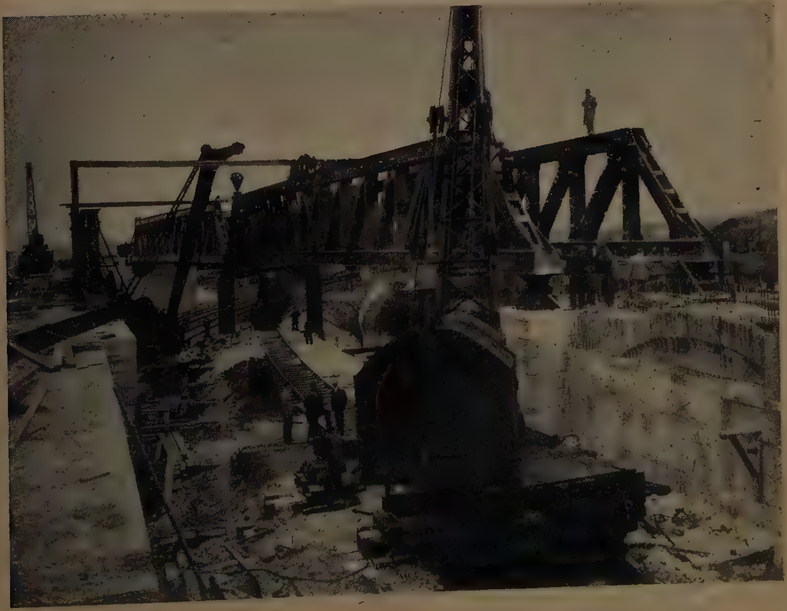
Moving Main Girder on Pollen Wagons

Fig. 9



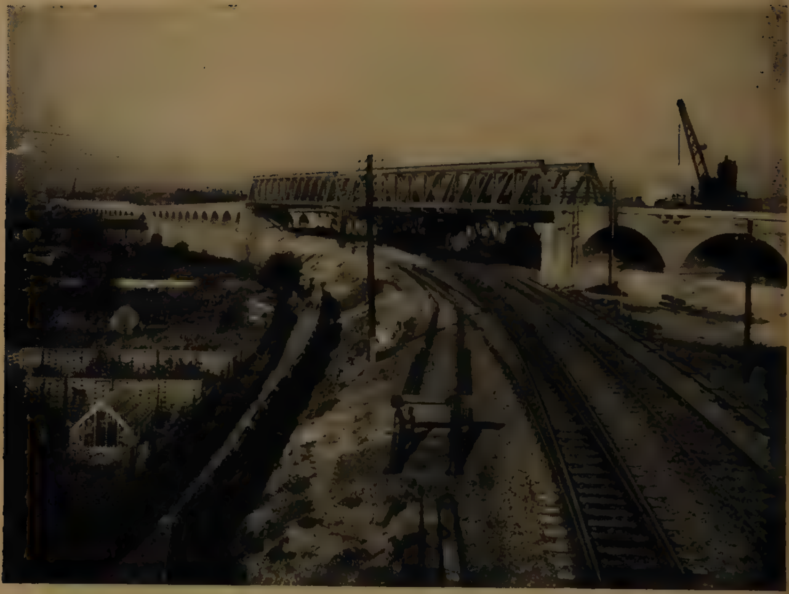
Landing Main Girder—One End steadied by Crane during Movement outside Radius

Fig. 10



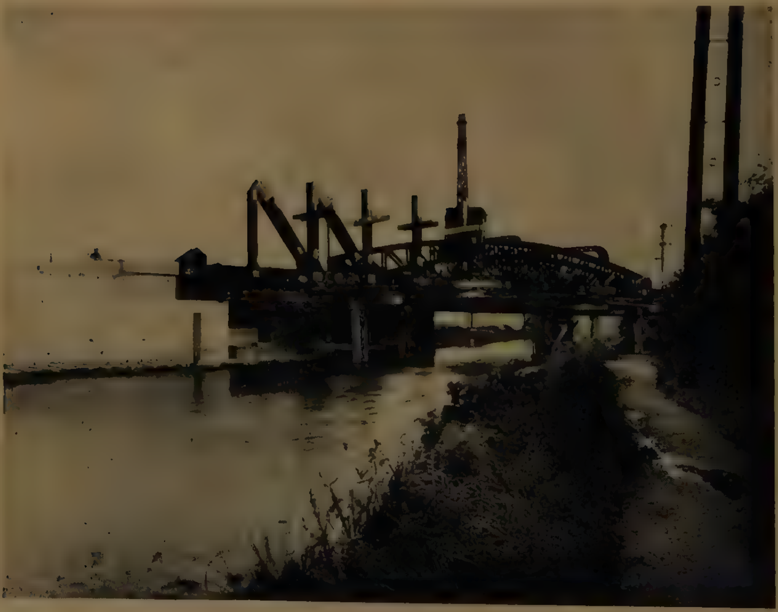
View showing Temporary Bracing of Main Girders

Fig. 11



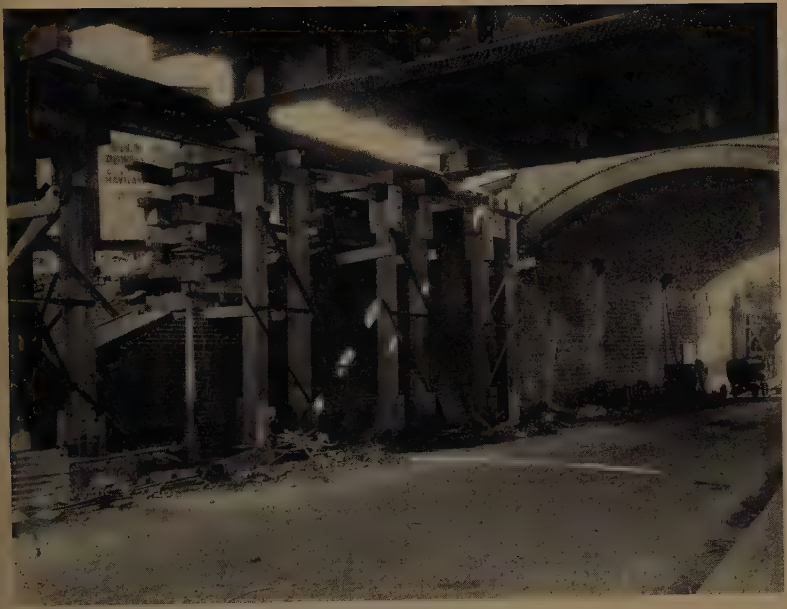
General View of Bridge "B" with Bridge "A" in Background

Fig. 12



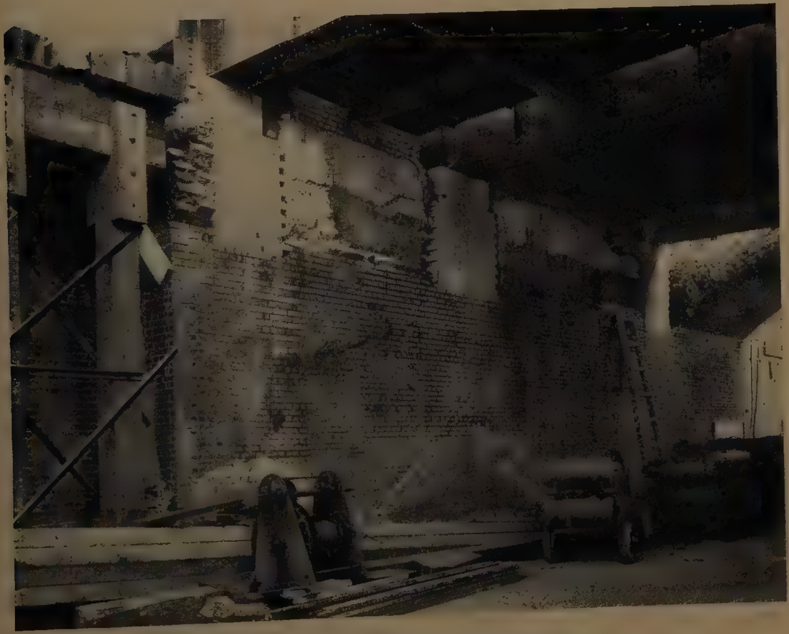
Bridge over Greenford Canal—Erection of Main Girders on Falsework

Fig. 13



Oldfield Lane—Before Rolling-in (note windows in arch)

Fig. 14



Oldfield Lane—After Rolling-in

(3) *Rolling-out the Old Structure and Rolling-in the New*

This method is often chosen when the total length of any one possession of the line is of paramount importance and when the temporary restriction of speed must be removed very quickly, but three possessions are needed altogether—one to land the new steelwork on trestling outside the bridge, one for rolling out the old work and rolling-in the new, and a third for loading the old work on to wagon after stripping preparatory to loading. The essential condition is that the site shall offer a clear width on one side of the old structure, sufficient for the new bridge, and on the other side sufficient for the old bridge when rolled-out clear of the new bridge.

A curve in the alignment of road or river under the bridge and near to it may make the temporary work complicated and therefore expensive, and although a gradient on the roadway underneath may decrease the size of trestles needed, it may also mean closing the road once there is a structure spanning the trestles.

It must be remembered that the work involved in erecting the bridge on its temporary trestles is the same as a similar permanent erection in method (2) above, but any concreting or waterproofing needed after erection of the main framework of the bridge can be carried out without disturbance to traffic, without overtime rates, and with due care; but although the track need only be broken once, the total cost of the method is on the high side because of the temporary work needed.

The temporary trestles should cover the whole length of movement, both to avoid the delay involved by abutment work and because it is essential to ensure a really good alignment on each rolling rail and that the rolling rails are parallel to each other—a condition worth checking by theodolite.

The usual rolling path is formed on each trestle top by a good serviceable rail (free from cripples and laid on its flat). After the check for alignment, the rail should be spiked to the timber supporting it. The girders then sit on short lengths of rail which ride on 2½-inch-diameter steel balls, running in the web of the continuous rail and fed from front to back of the carriage as the rolling progresses. Hand winches suffice for power and the travel on a well laid track is so easy that a preventer of cordage block and tackle is desirable with secure stops to prevent over-travel.

Before the day of the rolling-in, the new structure should be adjusted on its carriages so that it is exactly parallel to its final alignment and, following that, the roller path is marked out at 6-inch intervals generally and in inches for the last foot or so.

Provision should be made beforehand for jacks to lift the structure and remove the carriages and then to lower-out on to the final bearings, and the height of roller path must be set so that it is below the new work when thus lowered-out. It is advisable to have traversing jacks available to deal with any final adjustment of alignment, and jacking brackets fixed beforehand to the girder ends save time. Save-alls should not be omitted.

Nowadays trestles are built of the standard steel trestling remaining fro A.R.P. stocks and similar to the war-time standard military trestlin This is easy to handle, has good bracings, and is entirely recoverabl Although it can be manhandled, a light mobile crane on the roadwa below facilitates the work greatly, particularly in moving sections from roadside dump when the road is still open to traffic.

When a concrete underbridge is to be rolled into position at a site wher occupation is short, the Author considers that valuable time could be save in erection by limiting work on the abutment to a clearing away to a give level, such as the underside of the old bedstones, and casting the new structure with monolithic bearing blocks to sit at that bed level.

(4) *Combination of* (1), (2), *and* (3)

It is usual in such a choice to roll-in the new work and so gain the benefi of easy completion of the waterproofing, etc. It is very useful in dealin with reconstruction of a multiple-span bridge where, for operating reasons one or more lines must be kept open to traffic.

An interesting example of rolling-in only occurred at Greenford during the North Acton–Ruislip quadrupling. An arch span of 30 feet carrie the up and down mains and platform lines over the public road at Oldfiel Lane and was to be replaced by a steel bridge of 45-foot span with a new abutment at the western end. The demolition of the arch and the rolling-in of the new superstructure for three tracks (the up and down mains and the up platform line) were programmed as stage 1, in which all traffic would run on the down platform line. The steelwork for the three lines was erected complete with waterproofing on trestles to the north of the bridge when the new western wall was nearing completion.

At the same time, at the eastern end, a trench, 6 feet wide and spanned by lacing rails, was carried down to the arch backing and vertical holes were sunk through the arch backing and rings to springing level. In these, new brickwork was built to carry the new main girder bedstones which were set, and the rolling-in track on this side was laid over the new work.

The first and second stages of demolition were separated for safety by a longitudinal cut through the arch made in trench during traffic and a temporary timber spandrel was fixed to the portion of arch forming the second stage of demolition. (See *Figs 13* and *14*.)

During the main demolition, to avoid under-cutting the rolling-in track, a clean cut was made through the arch and its backing at the face of the eastern abutment by two Siskol cutters, and this cut was maintained about 6 feet in advance of the breaking-up work which was carried out by stripping the concrete with jack hammers, drilling holes through the arch at about 3-foot centres, inserting plugs and feathers, and driving home the plugs with jack hammers. Each of the Siskol cutters maintained an average rate of 2 hours per square yard of cut and the time required to drill five rings of brickwork was 2 minutes.

Demolition for Stage 2 was very similar to Stage 1 but two cranes were used to deal with the last main girder and the floor steel, by piecemeal erection.

The following times may be of interest :—

Stage 1

Saturday, April 22*nd.*

2.00 p.m.	Possession of up and down main lines taken.
	Removal of track and temporary beams begun.
2.45 p.m.	Started removing filling over arch.
4.10 p.m.	Started drilling the arch.
10.30 p.m.	Possession of platform line taken.

Sunday, April 23*rd.*

7.34 a.m.	Arch demolition completed.
7.45 a.m.	Rolling-in commenced.
12.10 p.m.	Superstructure jacked down and rolling track removed.
10.00 p.m.	Ballast walls completed.

Monday, April 24*th.*

12.50 a.m.	Normal traffic working resumed.
5.30 p.m.	Roadway under bridge opened to vehicular traffic.

Stage 2

Saturday, May 6*th.*

9.30 p.m.	Possession taken of down platform line.
10.00 p.m.	Commenced removal of filling over arch.
10.45 p.m.	Temporary beams removed.
11.00 p.m.	Commenced drilling on arch.

Sunday, May 7*th.*

12.07 a.m.	Siskol machine commenced.
3.35 a.m.	Demolition of arch complete.
3.50 a.m.	Possession of down main taken.
9.25 a.m.	All steelwork erected and riveting under 4-foot way commenced.

The Problem of the Centre Girder

In dealing with the reconstruction of a double-line through bridge with centre girder, there is always the problem of a temporary slew if traffic will not permit the whole job to be finished in one weekend. If new and old bearings are at different levels, the problem is further and sometimes needlessly complicated. If the condition of the existing centre bedstones permits their re-use, this is the greatest help the designer can give to the

man at site, but every effort should be made to obtain sufficient time f
the whole job to be carried out in one stage if the request to the Operatin
Department is a reasonable one. Using the piecemeal method, the secon
and third main girders must be landed in temporary positions on the abu
ments before the second half of the old structure is removed.

SINGLE-LINE BRIDGES

When single-line bridges are quite small, so that one crane from o
end can handle the greatest lift, no special problem arises. If two cran
are needed, room must be found on the piers or abutments to receive th
two new main girders outside the existing bridge before it is taken dow
and the old main girders must be similarly placed outside the new wor
before they can be loaded on wagon.

If the structure to be dealt with is a large one involving a number
spans, special measures are justified—as in the provision of an erectio
wagon for the approach spans of the Royal Albert Bridge, Saltash.
Here the main girder spans ranged from 70 to 90 feet, and at the Cornis
end an average time of 4 hours occupation of line was achieved for th
renewal of two main girders.

OVERBRIDGES

Overline bridges present few problems different from those mentione
above for underline bridges. If the superstructure carries railway ove
railway, the main girders can be handled from below in smaller radiu
than from above, but the flooring members are reached more easily by
cranes on the high level.

Where a road bridge crosses the railway, it is often difficult for the crane
despite a swan-neck jib, to place cross-girders because of fouling of the jib
on the main girder, and it is then helpful, after landing the main girder
remote from the side from which the crane is working, to place a temporary
beam across the lines a little above the level at which the cross girders sit
and inside the final position of the second main girder, and then to land the
floor so that one end is on the main girder and the other on the temporary
beam. The second main girder can then be placed by crane to receive
the outstanding floor and so be landed on its bearings. Folding wedges
enable the temporary beam to be dropped clear.

RENEWAL OF TRESTLES

Three-span bridges carrying roadways over the line, where in cutting,
form quite a high percentage of overline bridges in the Western Region,

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[1] H. D. Smith, "Reconstruction of Liskeard Viaduct and Scheme for Reconstruction of the Approach Spans of the Royal Albert Bridge, Saltash." Min. Proc. I.C.E., vol. 230 (1929–30, Pt. 2), p. 100.

and the design of the new work frequently employs pre-cast concrete trestles which are quick in erection. When a new trestle foundation is to be provided, the possession required can be reduced further by varying the spans so that the new foundation can be included in the preliminary work.

Similar attention to erection is needed in trestles under the line. The Author has in mind a single-line bridge over a canal now due for renewal. The new design shortens the spans and remedies badly moving abutments. It includes a new reinforced-concrete trestle on piles in the waterway, and here an inexpensive lengthening of the trestle cap will enable the main girders to be handled without expensive falsework.

PREPARATION OF THE ERECTION PROGRAMME

The erection scheme for the superstructure is prepared by the District Engineer and his staff. A steel-tape survey is made by an assistant, showing the abutments at bearing level, existing girder bearings, bedstones, running rails, and such temporary beams as will still be in place during the reconstruction. This should preferably be plotted at a scale of ½ inch = 1 foot. Where the track is straight, the new superstructure and bearings can be drawn on this plan. If the track is curved, it is advisable to draw the new work on an overlay and then prick through in the best position. Thus any discrepancies in the original survey work are found and account is taken of any error in abutment line or slewing of track since the first survey was made for design purposes, which may have been some years previously. It is also seen clearly whether any temporary work will foul the new work or hamper crane working.

During the making of this survey at site, much other information is collected and the following points should always be covered :—

(1) The maximum amount of preliminary work that can be carried out either without occupation of line or in a special small occupation. Both removal of old work and erection of new must be considered in this. Thus an unusually great depth of ballast, tarmac, or concrete may justify substitution of timber packings. Long temporary beams may be replaced by short ones spanning only from the new wall to the back of the existing bridge, and during this possession some of the new abutment work may be carried out.

(2) The nearest position to the face of the abutment that the leading axle of a crane can take and the closure rails needed for that.

(3) Whether obstacles exist to the free working of the crane jib or tail.

(4) How the crane can be watered. Pumping from a stream or provision of a tender may save sending the crane away to

water at a busy stage in the work and into a heavily lade
traffic section.
(5) The height of lift for overline bridges.
(6) Whether holding-down bolts hamper the free removal of the ol
work.
(7) Whether the site lends itself to rolling-in. The survey can b
extended for this when the method is settled.
(8) Whether the existing bedstones can be re-used or the new one
be fixed before the reconstruction.
(9) Whether the final waterproofing and abutment work is likely t
need a subsequent occupation, and if so, what temporary work
quick in removal, will be needed to carry the track over the
new deck and the trench at the abutment.
(10) The order of erection and loading of material.
(11) Whether room exists without staging to take a concrete mixer
portable generators for lighting, a compressor, or other needed
plant.

Before finally choosing the method, the Service Time Table is studied to see what occupation of lines can reasonably be expected and whether diversion of freight trains or the running of buses will be involved. This aspect of the work calls for a broad outlook from the Engineer, for if the most economic method from the engineering point of view causes avoidable traffic expenditure, disturbance of the travelling public, and delay to freights, the overall result may be extravagant. Working in stages or spending money on temporary works may then be well justified. The final plan and its repercussions upon traffic should be agreed early between the District Engineer and the District Operating Superintendent so that the promised facilities can be known to those planning excursions and other additions to the booked service which so often whittle down anticipated margins. The date must also avoid nearby re-laying and be suited to availability of crane power and special wagons.

The final detailed programme involves the appreciation by all concerned of every little operation in the work and the noting of those that can proceed concurrently. The time and labour required are thus known and the Operating Department can be given the required train arrangements for cranes and materials, the formation of the trains, and the necessary shunting. A meeting at site of representatives of all departments concerned should precede the work so that any doubts may be cleared up and close liaison between the District Engineer, his assistant for the particular work, and the Bridge Inspector is necessary throughout the planning.

The working positions of the cranes and their radii are decided either by direct measurement at site or by plotting on the survey, and these are marked out on the ground beforehand and any necessary packing timber is provided for the extension girders of the crane before the work starts.

Bearings and Bedstones

When a girder has a flat bearing plate sitting directly upon its bedstone, its landing is a speedy operation and the stone can be set with sufficient accuracy by plotting on the survey and steel-taping its edges from the quoin or the edge of an existing girder. Thus the mason can be given beforehand two nails to mark his stone position and one nail 2 feet or so below his finished level. The new stone can be marked in paint on scribed lines to indicate the bearing plate, and the setting of the girder can be quickly checked by a string line between pegs set beforehand.

When, however, the new bearing plate is a small one, resting on a bedplate with dowels or holding-down bolts, the setting of the stones must be triangulated by steel tape during their setting which will ensure accuracy to within half an inch. This interval for checking tends to reduce the tempo of the work and the Author suggests that the bearing plate directly on the bedstone is sufficient for spans up to, say, 60 feet. He has never known corrosion of the bottom flange at the abutment to be the criterion for a reconstruction.

The designer can be most helpful to the man at site by re-using existing bedstones wherever their position in plan and their condition permit. If the stone is too low, a packing of the needed height is called for at the new bearing plate, and if the stone is too high, the new steelwork can be detailed to bear at a level above its general line of bottom flange. The shop work thus involved is much less expensive than the renewal of bedstones capable of indefinite further life.

Pre-cast Concrete Work

The general run of pre-cast concrete work is pleasing to handle at site but, particularly in underline bridges, the Author thinks that designs should aim at producing structures requiring as few lifts as possible, so that erection may be speedy and full benefit can be obtained from the capacity of cranes and special wagons.

It should also be remembered that timber shuttering cannot be maintained to a perfect plane and, consequently, room must always be allowed for the " growth " of pre-cast structures where large planes are in imperfect contact. This is apt to give trouble, particularly in designs where the form of an earlier type of construction—timber, or steel—is reproduced in concrete. This question of warp and creep will also need the most careful attention in bridges where post-tensioning is to be applied, and the Author thinks that standardization of design, so far as it is ever applicable to bridgework, will produce the final solution, for repetition will justify the use of steel or other metal shuttering of rigid construction and, by increasing speed of depot output, will help in the overtaking of arrears.

The period required for the full completion of a structure can be greatly increased because of quite small quantities of in-situ concrete needed in th surface to which waterproofing is to be added, since the standard two-coa membrane system of waterproofing cannot be laid successfully upon we or even green concrete. A revision of deck design, an alternative water proofing system, or an additional membrane laid between the waterproofin and green concrete appears, therefore, desirable.

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The Paper is accompanied by fourteen photographs from which the half-tone page plates have been prepared.

Discussion

The Author introduced the Paper with the aid of a series of lantern slides.

The Chairman said that he thought that the Paper was a most valuable one from a practical man who dealt with the subject in a practical way. His experience enabled him to know just how important it was that the "man in the office" should appreciate the difficulties of the "man on the job." The Paper was extraordinarily useful, particularly for younger men who had not been able to get practical experience.

The Railway Division had stipulated that a District Engineer should present the Paper, and although Mr Butland was now Assistant Regional Engineer of the North Eastern Region, it was known that he had had long practical experience as a District Engineer; the Paper would be of use for many years.

Mr M. A. Henry said that the Paper was particularly interesting to him since it dealt with the practice on the Western Region. In addition, he had had the pleasure of being associated with the Author in 1938 and 1939 in the interesting and extensive works described in connexion with the erection of two very heavy spans at Greenford, as well as the bridge reconstruction work at Oldfield Lane.

The Author had referred to the use of brick-faced concrete for abutments and wings. Mr Henry was in full agreement with that form of reconstruction, and mentioned that in the case of a recent renewal of a large wing wall at Skewen on the South Wales main line that form of construction had been adopted. It was an original Brunel viaduct and the old wing, as well as the whole of the viaduct, had been built to very slight dimensions. The existing masonry had been re-used, the balance being made up by the purchase of similar stones from a nearby quarry. The old wall had been bulging to a versine of about 18 inches in its length, and had been very weak indeed when the remedial works were taken in hand. The work had involved all the problems associated with similar jobs of that nature and had had to be carried out in short lengths. That had necessitated some heavy trench timbering and also the provision of substantial raking struts to ensure stability of the old wing. Carrying beams had been provided under the adjoining track while the work was in progress. The height of the wall was approximately 35 feet, and the result was a wing entirely in keeping with the character of the original structure.

Mr Henry referred to several cases which illustrated methods employed on the Neath District. The question of suitable and available crane power always had to be one of the primary considerations in bridge erection. The method of rolling-out the old and rolling-in the new structure was not new, but although it was in favour at present, experience had shown that there was no saving in costs, but rather the reverse.

On the Neath District it had been formerly the general practice to carry out the work piecemeal, and in many cases frequent occupations of running lines were involved. The perfect waterproofing of a bridge deck was never carried out without difficulties and in some cases the work was not entirely satisfactory. In the rolling-in method there was usually ample time and opportunity to carry out that important feature in a first-class manner and that, in his view, was one of the main points of advantage in the method.

The design of bridges as now carried out at headquarters took full account of subsequent maintenance, a feature which had never been fully appreciated in the past. A bridge at Sarnau, about 4 miles west of Carmarthen, carried the South Wales main line over the London–Fishguard trunk road, A. 40. The old bridge, a three-girder structure with trough flooring, was strengthened by relieving girders, but the ironwork had been deteriorating rapidly and reconstruction had become urgent. The new bridge was also a three-girder structure with a floor of cross-beams with pre-cast permanent shuttering and in-situ concrete filling, waterproofed by the standard method. The rolling-in method had been adopted, two roller paths being used, supported on military steel trestling. Two 2-ton-capacity winches were used for hauling the bridge on the roller paths and six 100-ton-capacity hydraulic traversing jacks were used to lower the new bridge on its bearings. The roller paths and carriages were fabricated by the Neath bridgemen.

The carriages had consisted of two 8-foot-by-6-foot BS beams linked by steel plates, 21 inches by ½ inch, welded to the top and bottom flanges. On the underside plate two lengths of rail, bent at each end, were welded to form the inverted roller path, and on the top plate two pairs of ang cleats were welded to enable the grooved timber seating blocks to secured to the carriage by coach screws.

Owing to the volume of main-line traffic and the design of the old an the new structures, it had been necessary to carry out the erection in on operation. The occupation of both lines for 12 hours had been arrange commencing on the down line at 5.20 a.m. and on the up lines at 7.20 a.m Rolling-out of the old structure had occupied 23 minutes, and the pulling-i of the new had taken 90 minutes, the latter operation being completed a 11.15 a.m. Three men were employed on each winch; the carriage travelled on 3-inch-diameter steel balls, and the work had proceede smoothly throughout.

The sequence of operations was: removal of track; jack-up ol structure and place carriages under; roll-out old structure; roll-in ne structure; withdraw carriages; jack down; fix on bearings; reinstat track. The work had been completed in 10 hours, with a speed restrictio in operation, the bedstones having been set in position on a previous Sunday. The whole of the steelwork, except the main girders, was encased in con crete, which should eliminate some future maintenance. The bridge ha a skew span of 50 feet, the angle of skew being 46 degrees, and the ne structure weighed about 220 tons.

Mr Henry expressed a preference for leaving the underside of cros girders exposed, to enable any leakage or other defect to be more easily observed by the bridge examiner.

A bridge over the River Taf on the Pembroke and Tenby branch lin had been dealt with in very much the same way. In that case the underside of the cross girders was left exposed, thereby enabling the bridge t be maintained with the minimum of inconvenience. A train had been run over the new bridge 3½ hours after occupation.

The Author had mentioned a very important point on p. 741 under the heading, "Bearings and Bedstones." The precise and exact positioning of dowelled bed plates in a rolling-in job very often meant the difference between success and failure. These refinements for spans of up to 60 feet could, in his view, be eliminated with advantage.

Mr Henry then showed a number of slides illustrating bridges at Sarnau, Whitland, and Margam.

Mr J. S. Campbell observed that the Author had said that the preparation for every bridge reconstruction must be carried out in a careful, detailed, and pre-planned manner right from the matter of surveying down to the detailed erection scheme. He agreed that it was the best way in which a scheme could be translated into fact, since the various factors in bridge building—from design to erection—had a reflexion in some form or other on the type of design produced. Difficulties experienced in the erection of certain structures should be noted for consideration in subsequent designs of a similar nature.

He had had experience in outside industry, as well as on the railways, and he had always found it the best arrangement for the bridge office to be responsible for the whole of the bridge work. There was a great danger, found very clearly in outside industry, of people getting into "watertight compartments." He felt that the best job could be produced when the bridge office was completely responsible for the whole of the work—abutments, superstructure, and erection. So many difficulties arose in the erection of bridges that, if the designer were not acquainted with them, they would be repeated because he would have no cognizance of them. Mr Campbell knew that co-operation did exist—the Author had shown clearly that it existed in the Region in which he had once served.

On the question of the design and building of bridges, the maintenance aspect was, in Mr Campbells' opinion, all-important. He thought that most bridge engineers would agree that it was not good policy to have open steelwork bridges, especially for railway purposes. Even if they were used in branch lines there was a serious difficulty because usually the places were difficult of access and it was not desirable to take maintenance gangs or painters along there very often. He suggested that one of the most important points to be borne in mind was that excessive maintenance costs should be avoided.

The Author had suggested that there were certain difficulties in setting the superstructure of a bridge on to the abutments where the bedplate had to be bolted. He admitted that difficulty, but suggested that it was essential to do it even on small bridges. If a small span was allowed to float, the full impact on a very rigid structure would cause deterioration at the ends of the superstructure which might well lead to serious difficulties in the future.

Mention had been made of the rolling-in method of bridge erection. He agreed that that was the best method, even where it involved extra cost. He had carried out a survey of about 200 bridges with a view to listing them in sections to show the effect of approximately 10 years of life on the condition of the deck. They were divided into sections—those that had been rolled-in and those constructed on the track. It was remarkable that the record showed that there was very little cracking in the decks of the rolled-in bridges except on severe-skew spans. He would say that wherever practicable and where the joists were encased in concrete then the bridge should be rolled-in.

On the question of the level of the bedstones, there might well be an argument between the man responsible for the abutments and the man responsible for the superstructure. If in the design stage the ends of girders were cut to accommodate the level of existing bedstones, that might lead to serious difficulties. For instance, if the bedstone was 2 feet too high that might mean a square-cut corner in the girder, and he felt that that was a matter for the bridge office to decide. The girder was made in the shop and it was probably easier to handle than the bedstone on the site. If it was cut, there was the possibility, as had been proved by tes of serious previously unknown developments occurring upon applicatio of the load.

He called attention to the type of design discussed when Mr J. S Robertson had given a commentary on a film (shown at the Institutio before a joint meeting of the Railway Engineering and Works Constructio Division) illustrating the replacement of the Scottish bridges after floo damage. The bridge deck had been a clean-cut type of structure, wit welded main girders braced together. He thought the Author would hav been delighted to handle the erection of jobs of that type. It was type which he thought ought to become popular from the points of view of both the designer and the erector. It should also become popula because it was a good job from the maintenance aspect. All those factor were seriously considered and, up to the present, none of the promise which the designer laid down had had to be withdrawn.

He joined with the Author in hoping that there would be furthe standardization of bridges to make it easier for the man on the job who had to erect them. There were in existence standards of underline bridges from 10 feet to 46 feet span in reinforced concrete—they were, in fact used throughout British Railways. The adoption of pre-stressed concrete instead of reinforced-concrete slabs would effect a saving in constructiona depth. The average type of bridge could be made today of the standard welded type of girders, outer and centre, together with the pre-stressed concrete deck, and these bridge components had been given consideration in relation to erection and all the other difficulties which the bridge builder had to face.

The difficulties which traffic imposed upon the railway bridge engineer were being overcome very skilfully. He felt that the practical man was able to prevent the "man in the office" from developing a narrow outlook particularly where methods of construction were concerned, and concluded by quoting from the Press: "We bridge engineers must keep our mind in harmonious accord with the changing circumstances and events as they are affected by modern engineering."

Mr P. S. A. Berridge said that after spending 20 years on an Indian railway where total occupations for bridge reconstruction never exceeded 3½ hours, he was curious to know why total occupations for similar work in Great Britain were so very much longer. He put forward the following explanations and exhibited several lantern slides. First, on the North Western Railway (India) bedstones were cast in situ, generally before the renewal of the superstructure, and a "dead-stop and 5-miles-an-hour" restriction was enforced for 4 or 5 weeks. On the Great Western Railway, on the other hand, bedstones were pre-cast and were often set in position during the same occupation in which the new span was erected. That prolonged the occupation, but reduced the time during which the speed restriction had to remain in force.

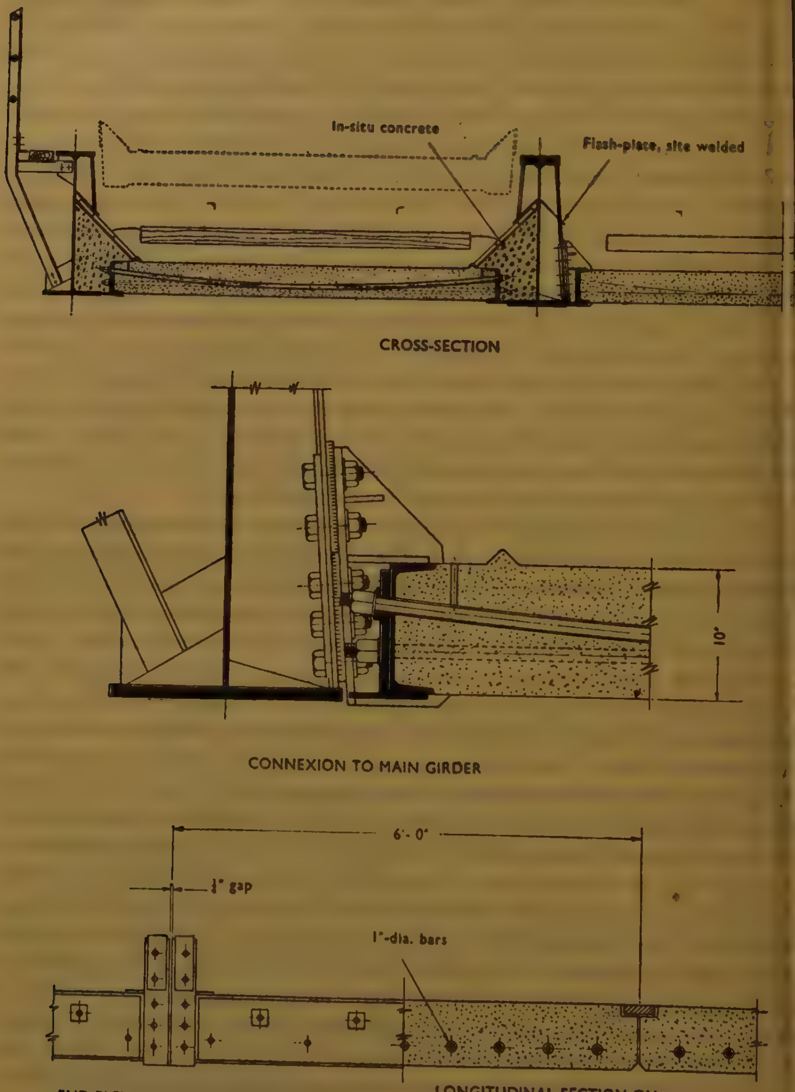
Secondly, the Indian railway was equipped with cranes specially built for bridge work. *Fig. 15* (facing p. 750) showed a 25-ton bridge-erecting crane. There were two of these cranes on the North Western Railway and they could lift and travel with 12 tons at a radius of 47 feet 3 inches—sufficient to enable a 40-foot span to be placed in position ahead of the buffers. By changing the jib for one 82 feet long, one of these cranes could be used to pile ahead of a 40-foot opening while the other crane with the standard jib could place the girders in position. *Fig. 16* (facing p. 750) showed one of these cranes fitted with an 82-foot jib. On British Railways, the bridge erectors had to make do with breakdown cranes which were comparatively slow and cumbersome, and had not nearly such a good reach. After making due allowance for the gauge on Indian railways being 5 feet 6 inches he was nevertheless convinced that cranes of a similar capacity could be designed for bridge erection on the narrower track of British Railways where they would fill a pressing need.

In India, bridge decks were never waterproofed, whereas in Great Britain nearly all spans were waterproofed—a process which was costly in time and money and often involved an extra Sunday occupation. Also, in India train services were generally the same every day of the week and the workmen received no extra pay for Sunday work.

The Author had rightly called for a reduction in the number of crane lifts, but Mr Berridge would go further and eliminate all site riveting and welding during erection. It was with this object in view that the prefabricated primary- and secondary-unit deck had been evolved for through-type spans. *Figs 17* and *18* showed the erection of such a deck. After setting the main girders in their final positions on the bedstones, the primary units were landed on shear plates on the tapering face of tee-shape stiffeners and the cross girders were secured to those stiffeners with high-tensile-steel bolts. The secondary units dropped in between the primary units upon which they rested. The arrangement did, however, involve three separate lifts for the secondary units in the width of the deck and although assembly was quicker than in the case of the jack-arch deck, he agreed with the Author that it would be preferable to eliminate the use of small units in the deck. A development of the primary- and secondary-unit deck for through-type spans was the pre-cast pre-stressed concrete unit deck, illustrated in *Figs 19*. That further reduced the number of lifts and had the added advantage that the pre-stressed concrete did not have to be waterproofed all over. But because of the need for a high degree of accuracy in the fitting of the concrete units to the main girders, that type of deck was introducing a further complication in the contract for the supply of the bridge. The pre-cast pre-stressed concrete units had to be included in the contract for the supply of the fabricated steelwork in order that the whole span could be assembled in the makers' works before acceptance.

The Author had referred to the difficulty of siting bedplates accurately

Figs 19



CROSS-SECTION

CONNEXION TO MAIN GIRDER

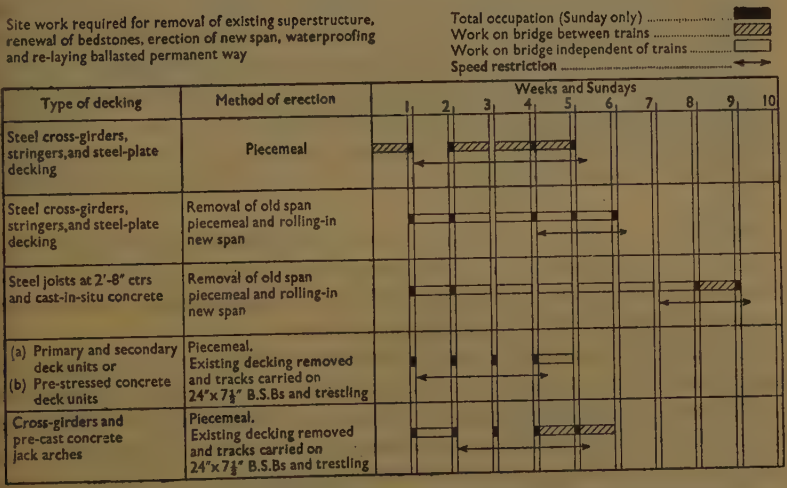
END ELEVATION OF DECK UNITS

LONGITUDINAL SECTION ON CENTRE-LINE OF DECK UNITS

PRE-CAST PRE-STRESSED CONCRETE DECK UNITS FOR THROUGH-TYPE SPANS on the bedstones. Stepped bearings were obviously an improvement on the old flat plate bearings which used to " work " on the stones. The difficulty was overcome by temporarily welding the bedplates to the bearing slabs with small angle cleats before erection. In that way, the bedplates landed with the main girders and locating them was as simple as it had been with the flat bearings. The angle cleats were knocked off afterwards to free the bearings.

Recent deck-type bridge designs were such that the pair of main girders were braced together before erection and the decking consisted of pre-cast reinforced-concrete well-deck units which were landed on a layer of concrete secured to and pre-cast on the top flanges of the main girders beforehand.

Fig. 21



Occupations and Speed Restrictions for the Reconstruction of a Double-Track 3-Girder 40-foot Through-Type Span

Holes were provided in the well-deck units, fitting over bolts attached to the main girders and protruding through the layer of concrete ; the bolts acted as dowels and were subsequently grouted-in to hold the deck in position. The layer of concrete on the girder flanges was cast in short lengths with spaces between in order to avoid any likelihood of the concrete shearing off the flanges. *Fig. 20* showed the erection of well-deck units.

Fig. 21 had been taken from a Paper presented before the International Association for Bridge and Structural Engineering. It showed the extent of the total occupations and speed restrictions likely to be required for various forms of construction and different erection schemes for a three-girder 40-foot double-track through-type span. The number of occupations and the length of time that speed restrictions had to be enforced varie according to site conditions. The cost of slowing-down and acceleratin trains on a main line was not easy to assess in money, but the effect of prolonged speed restriction could be to send up the fuel bill very considerably, to say nothing of the inconvenience to the Operating Superintenden if trains were not timed as fast as the available motive power permitted Piecemeal erection of a steel-decked span (the top one in the diagram) mean a speed restriction extending for more than 4½ weeks. Rolling-in the sam span cut the speed restriction to 2½ weeks and that would be further reduce to 2 days or even less if the old span were rolled-out as well. The fourt and fifth examples—the latest designs to which Mr Berridge had referred— would also require very short periods of speed restriction if the old and new spans were rolled-out and rolled-in respectively. And that method of erection would also reduce the number of Sunday occupations to three—one fo erecting the trestling, one for erecting the new span, and one for rolling-out and rolling-in and removing the old girderwork and the trestling.

On p. 732 of the Paper, the Author had referred to the need for care to avoid over-tightening the bolts securing lifting cleats to the top flanges of the girders. Those bolts would be in direct tension and, unless overloaded, the stresses in them during lifting would not be affected by the initial tightening.

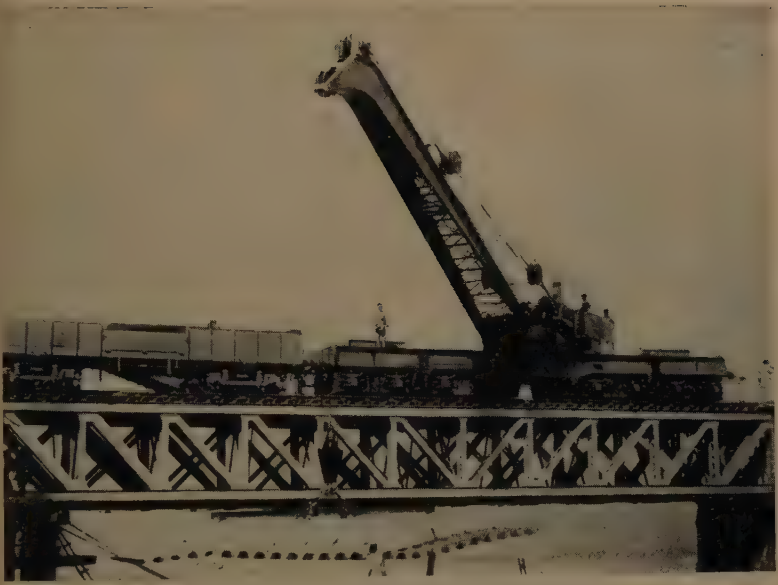
Mr W. E. Smith said that on the North Eastern Region, the same authority was responsible for the design and erection of bridges. The Bridge Office prepared the design of sub- and superstructures, the contract documents, and supplied the supervisory staff.

It was often possible to design identical bridges on more than one district and include them in the same contract. He referred to a current case of two bridges, the spans of which were identical, about 40 miles apart. They had been let as one contract which would be supervised by one man, who would make all the arrangements with the traffic people, the highway and other authorities, and the District Engineers.

The Author had referred to brick-faced and concrete abutments and wing walls. The practice in the North Eastern Region had been to provide courses with a lift of 2 feet 2 inches. The same type of construction had been used with a masonry face where, perhaps, they had been requested to do so by a County Council who were making a contribution towards the cost of the job. Mr Smith thought that the idea of the concrete-block skews was original—it was sometimes difficult to get skew bricks from the brickworks.

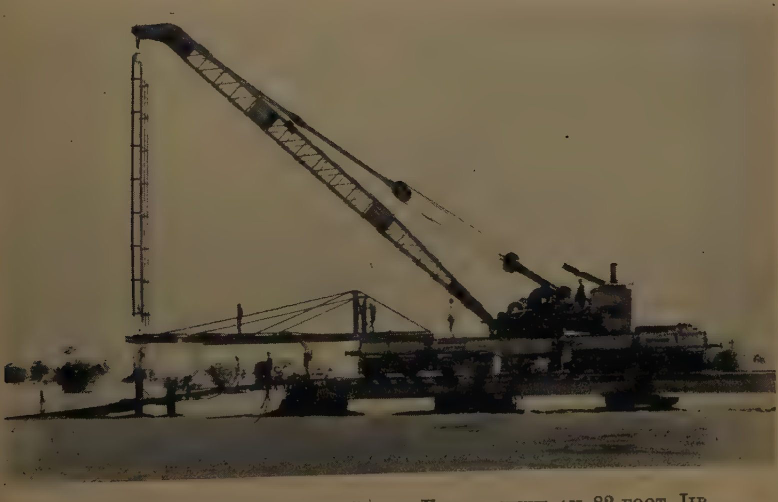
With regard to the provision of a new abutment, it was advisable to position it, if possible, behind the existing abutment and clear. Temporary superstructures carrying a track did not need to take any load in excess of " A " loading—that was 20 units at 3 revolutions per second—since the maximum loading for any type of engine was less than 18 units. There would probably be a speed restriction of about 15 miles an hour, so there

Fig. 15



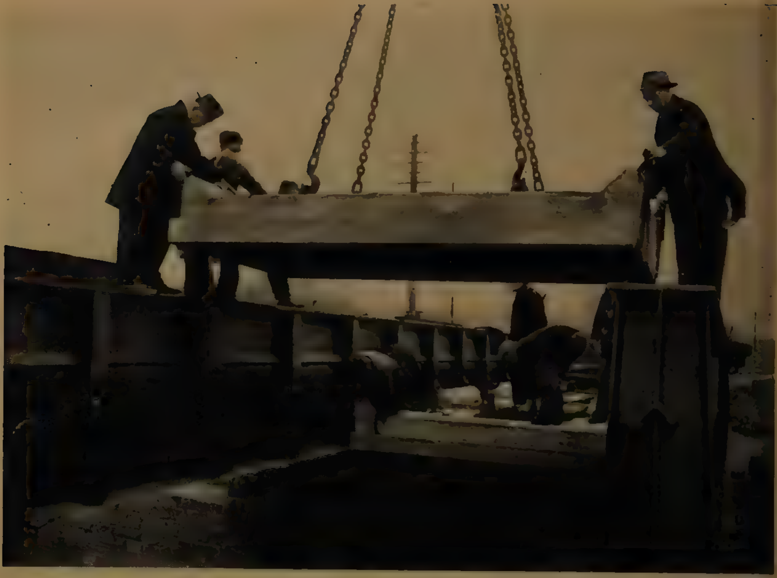
25-TON BRIDGE-ERECTING CRANE
(North Western Railway, India)

Fig. 16



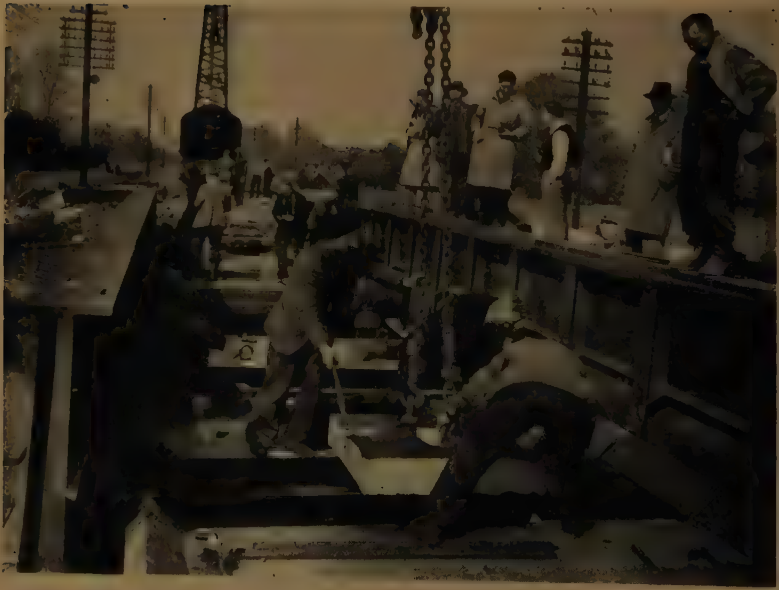
25-TON BRIDGE-ERECTING CRANE FITTED WITH AN 82-FOOT JIB
(North Western Railway, India)

Fig. 17



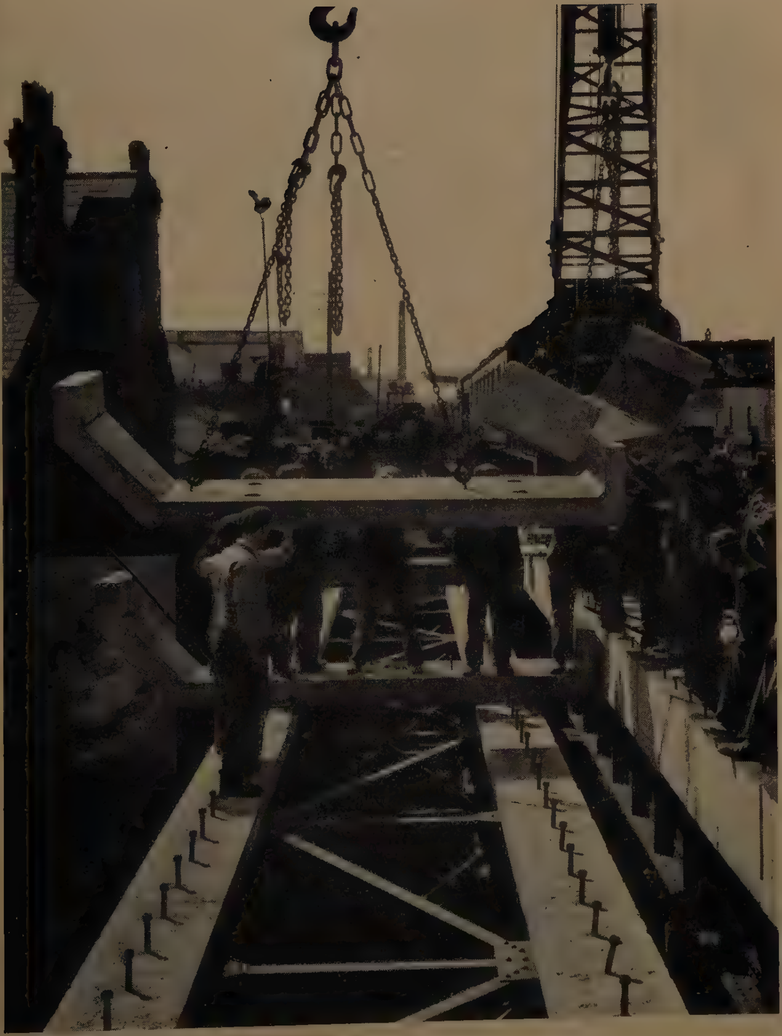
Placing a Primary Unit in Position

Fig. 18



Placing Secondary Units in Position

Fig. 20



THE ERECTION OF PRE-CAST REINFORCED CONCRETE WELL-DECK UNITS

was a certain margin to work on. If the old abutment would take the extra span, then advantage should be taken of that to take the temporary beams over the new abutment.

A rail-carrying gadget was in use which would span rather more than 6 feet and that allowed easy access to any work on abutments directly below. In longer spans, joists with cross-ties were used or, alternatively, pairs of joists with welded half-joists acting as crossbeams, supporting longitudinal timbers. They were available in units of various lengths and were found to be very useful. He had not met a case of a new abutment running through an existing one; in the event of major repairs to existing abutments, there should also be a substantial trestle in front of the structure.

With reference to the renewal of underbridges, Mr Smith said that in one case on the main line, a bridge of 43 feet span had been fabricated about 2 miles from the site; it had been transported on a wagon and put in position within an hour and a half of arrival. The job had involved rather careful gauging, there being only about ½ inch to spare on either side through various bridges. The old bridge had been taken out, the new one brought in, and the job completed within 12 hours.

A similar job nearby on a three-track bridge of about the same span had been brought in in units of 11 feet 2 inches width. It had been previously waterproofed except for a patch about 6 inches wide along the riveting line in the "sixfoots." The job, on electrified track, had been completed inside 24 hours.

In the case of a skew bridge, it was often easy to get the main girders in with a 15-ton crane. It had been found that the breakdown cranes were very slow, and he would not recommend their use unless there were no alternative. If it was a three-girder bridge, it was possible to utilize one track, off-load the new girders, and then return to clear off the old sections and put in the new bridge. It had been estimated that that type of operation could be done in about 24 hours.

For rolling-in he would not recommend the use of timber trestles. Experience had been that when the timber was wet, it warped and sagged, and it was most difficult to get the bridge moving.

Mr Smith went on to describe another example where two bridges, each of about 80 feet span, were being rolled-in within ¼ mile of one another. It had been intended to carry out both operations simultaneously but after being delayed for about 3 hours by an air raid, trouble was experienced with one of them. The roller paths were supposed to be high-tensile steel with the usual balls. Halfway through the procedure it was found that a stretch of mild steel had been inserted and the balls had cut into it. Fortunately, a crane was alongside. The girders were eased up, the balls removed, and a section of high-tensile steel from the bridge that had already rolled-in was inserted. The job was finished an hour and a half ahead of time.

With regard to single-line viaducts, there had been a rather interesting example over the River Eden. The viaduct consisted of 70-foot spans and sidings were available about 4 miles away from the bridge. The first consideration had been to make sure that the crane rope was of sufficient length and that the crane could be watered and fuelled. A period immediately after a full moon in June was selected, and although certain contractors had estimated 4 days, the engineers had completed it in 3 days. Utilizing all the hours of daylight, the new steel girders were first off-loaded (by means of two 15-ton steam cranes) on to timber cribs in the river bed —the water being relatively low.

After stripping the track, the old wrought-iron girders of the first span were off-loaded on to the river bed, the new girders erected, and the track made good. The cranes then moved forward, and the operation repeated for successive spans. On completion of the work, the old girders were recovered and loaded on to wagons. The point was that at 8 o'clock on the third night it had started raining heavily in the Pennines, and during the night the river rose by approximately 5 feet. If the contractor's time table of 4 days had been accepted, the girders would have been under water.

The Author had mentioned the trouble of erecting cross-girders between main girders. That trouble could be dealt with either by erecting a runway on the main girders and dropping the cross girders in between, or by providing longitudinal girders with jack-arching between them.

It might be found that four or five bridges of a similar type existed on the same branch line. There was no reason why, by careful arrangement, even up to four bridges could not be erected on one Sunday with two cranes. That had been tried on the Carlisle Branch. The bridges were about ¾ mile apart, and there had just been room to have the train marshalled between each pair of bridges and for the crane to work clear. The four bridges had been completed in a matter of 24 hours.

Mr C. W. King said that on p. 727 the Author had referred to the action which had been taken to ensure that the new design fulfilled the requirements of all users. Mr King could not see any specific mention of the action that was necessary to ensure co-operation between the bridge engineer and the permanent-way engineer to make certain that any necessary improvement in alignment or in getting a greater ballast depth were brought about. It was possible that on the Western Region those needs did not arise, but in some regions it was a point which it was most necessary to keep in mind.

Referring to the direct labour organization (mentioned on p. 727), he said that a similar organization existed on the London Midland Region, and he wondered whether that was an inheritance from the transfer of the regional territory. The point was of some interest inasmuch as the Western Region organization provided for a self-contained district set-up, whereas in the London Midland Region the District Engineers were not fully equipped to carry out all the bridge work. For that reason, on the London Midland Region the direct-labour organization under the jurisdiction of the Resident Engineers was a very useful adjunct.

He was rather disappointed that more had not been said in the Paper regarding the design of temporary works to enable traffic to run at higher speeds than 5, 10, or 15 miles an hour. On a line where the normal speed was 70 or 80 miles an hour, it was very bad from the operating point of view to have to come down to those low speeds, and from a departmental point of view it was often a severe sacrifice of opportunity to carry out re-laying or other works to have such speed restrictions during the time that a bridge reconstruction was in operation.

A lot of thought had been given to that problem on his region recently. In at least two cases speeds of up to 40 miles an hour had been possible. It depended very largely on what could be done to improve the temporary bearings of way beams. He thought that the problem of the stiffness of the way beams had been overcome. During the 1939–1945 war a number of bridges had been put up in a hurry—and some were still in existence—without any severe speed restrictions. That was because of the stiffening which had been put into the way beams. Those bridges were, generally speaking, on the old abutments. The difficulty arose when the way beams had to be put on to a clay foundation. There the soil mechanics people should devise some treatment similar to formation strengthening. Did the Author know of any way in which a further development could be introduced? As a member of the Regional Suggestions Committee Mr King was always hopeful that someone would suggest how temporary bridges could be constructed while maintaining train speeds at 40 or 50 miles per hour.

Reference had been made to the organization for the design and construction of bridge work. From experience he felt that it was very dangerous to generalize on any subject. The availability of labour and the accessibility of the site to headquarters were factors to be taken into consideration. But in his view, the real solution was for the head office to produce standard drawings and for the District Engineer to survey, prepare his work in detail, and carry it out, if for no other reason than that he must be responsible for the safety of the line and that it was far better that the erection should be in his hands.

Mr L. E. Hawkins referred to Table I and said that he was rather puzzled by the small expenditure on painting, and wondered if that figure meant that the cost of the painting was very low, or that there was a very long interval between the painting. Out of 1,500 bridges probably 600 or 700 were of metal, and if painted every 7 years, the average of about £28 a bridge seemed to be a very low figure.

He agreed with Mr Henry, though for different reasons, that the bottom flanges of steel joists (encased in concrete) should be left exposed.

He thought that the Author's system of brick-facing effected a vertical bond but not a longitudinal bond. Mr Hawkins said that he had found that a better bond was obtained by building the wall 14 inches and 9 inches thick in alternate lengths of 3 feet and staggering each 3-foot-high lift; that gave an average brickwork thickness about 2 inches less than that of the Author.

As opposed to the Author's experience, Mr Hawkins had found that corroded bearings were frequently one of the causes for a bridge renewal, and he had found old bridges particularly troublesome, where the bearings had been inaccessible.

The Author had said " A revision of deck design, an alternative waterproofing system, or an additional membrane laid between the waterproofing and green concrete appears, therefore, desirable." Mr Hawkins had had some rather unfortunate experiences with the membrane system and he was very much in favour of asphalt, not laid direct on the concrete but laid on to an intervening layer of bituminous sheeting.

The Chairman said that he was very pleased to see a reference made in the Paper to smoke plates. During his career he had found that the omission of smoke plates could cause a lot of damage, necessitating repairs in many cases, particularly of steel-encased or reinforced-concrete overbridges.

In his experience, brick-faced concrete walls had not always been satisfactory. He had been directly responsible for one built alongside a canal over a railway tunnel, and when a cofferdam was filled there was a serious run of water into the tunnel. It was discovered that the water had seeped through the brickwork and then gone down between the brickwork and the concrete on the cast-iron girders forming the roof of the tunnel. The fault was cured by waterproofing the face of the brick wall. The moral attached would seem to be to ensure good bonding between brickwork and concrete when that type of construction was employed. He agreed with the Author that availability of water for cranes was an important factor in both brickwork and permanent way work, particularly since cranes were used to an increasing extent and there were as yet very many more steam cranes than diesel-engined cranes available.

⁂ **Mr W. T. Wilks** said that the Paper should be very instructive to those Assistant Engineers and District Engineers who had limited knowledge of bridge construction. The procedure to be desired was set out, namely, that the District Engineer personally should direct the erection programme, but in many instances the whole matter was left to the Bridge Inspector and the Assistant Engineer to conduct the necessary correspondence between the co-ordinating departments. The Bridge Inspector drew on the knowledge obtained during his training, and that often resulted in the adoption of out-of-date methods and excessive labour forces. The sum total was extravagance and little progress in the development of new and more economical ideas.

⁂ This contribution was submitted in writing—SEC. I.C.E.

The methods of erection described by the Author had been in use for many years, and were based entirely on the utilization of railway travelling cranes which were not particularly economical. Mr Wilks suggested that, in many cases, it would be far more economical to erect near the site a modern crane with a long jib for "complete erection" and also "piecemeal" erection. A study of the method of erecting heavy loads at the new oil refinery at Fawley would be worthy of consideration by railway engineers, in order to stimulate ideas for more modern methods of bridge erection. Railway engineers should recognize that there were several types of cranes besides those used on railways.

Few people with aesthetic taste would agree with the Author's remarks about the pleasing results of mass abutments in brick-faced concrete as adopted by British Railways. The only difference between the style of today and that of 50 years ago was that engineering bricks had replaced blue bricks, and that might be because of the high cost of the latter brick. Pleasing results with bridge abutments could be obtained without much extra cost by careful consideration of the brickwork treatment, such as alternate courses oversailing by about ½ inch, or the choice of jointing and bond. With concrete throughout, there were several very effective ways of treating the surface which might be known to railway engineers. The aesthetic appearance of our railway bridges left much to be desired and Mr Wilks suggested that there was ample scope for improvement without incurring additional cost.

The use of pre-stressed concrete for temporary work in carrying the track over narrow trenches which had been excavated for abutments was bound to be an unnecessary extravagance when railways held quite reasonable stocks of plain rolled steel beams, which were as adaptable as any other medium. Would the Author elucidate the advantage of pre-stressed concrete for that particular job?

On p. 732 the Author had stated that, for comfort, the tension on the bolts for the lifting cleats should be 4 tons per square inch. How that working stress could be achieved by a spanner adjustment, or how it was related to comfort, was difficult to understand. If the bolts were "hand tightened," that was the obvious practical approach. However, Mr Wilks deprecated the fact that the Author even suggested bolts to be in tension, since it was a well-known principle in good bridge construction that bolts and rivets in tension should be avoided. The simple solution in the particular case mentioned by the Author would be to have a lifting bracket on the end of the girder with the bolts in shear—a method which was in general use.

Mr Wilks would direct attention to the Author's slide No. 5 of Penleigh Road Bridge, Westbury. It confirmed his observations about excessive labour forces to carry out constructional work on railways, and unfortunately a significant characteristic which contributed to costly work and the financial instability of British Railways. It would be noted that there were seven men actually working and seven looking on, and perhaps the other two were Assistant Engineers also employed on that site.

The Author, in reply to the discussion, said that one of his chief difficulties in writing the Paper had been the selection of examples. Those presented by the speakers, both with and without slides, had added materially to the value of anything that he had said.

He envied Mr Henry his recent experience with masonry work. It was so long since Mr Butland had had the pleasure of working with banker masons that he was certain that Mr Henry had been enjoying himself a great deal. He was a little puzzled by the carriage Mr Henry provided for his rolls. If he had a steel-to-steel contact, it might just as well be the main girder itself direct on a short length of rail. In fact, that was all he had ever found necessary. He agreed heartily on the question of covering steel, but perhaps in a rather different way. He thought that if it were possible to provide about 3 inches of cover, there would not be very much trouble. Of course, with pre-stressed concrete it was hoped that shrinkage cracks would not occur, but where the cover was normally thin or where the cover consisted only of small pre-cast sections which could not be in intimate contact with steel at all, then trouble would arise if smoke fumes could approach that area.

There seemed to be a difference of views on the question of bedplates and whether or not to use them. He thought that Mr Campbell was exaggerating when he talked of bridges wandering off the abutments. The Author knew of many smaller bridges that had no fastening whatever and that had behaved quite happily for many years. He thought that it was worth while going to any trouble to avoid (*a*) having to replace bedstones at all, and (*b*) complicating the actual fixing of a bearing during an occupation of the line.

He agreed that one could attach a bedplate temporarily to the bearing plate itself by welding and afterwards cutting the weld. But, even then, a dowel still had to be inserted. It was necessary either to have a very big hole through the bedplate in order to drill through into the stone underneath, or to set the work with just that niceness which tended to slow down the tempo of the job. When he had referred to differences in level, he had not for a moment had in mind a difference like 2 feet but rather one of the order of 1 or 2 inches. He thought that if Mr Campbell had to deal with a bridge where the bearing was 2 feet higher than the level of the bottom flange he would use the type of girder which could be seen in many viaducts where there was 4 or 5 feet of narrow parallel work at the bearings followed by an inclined bottom flange perhaps 8 feet long and then another parallel portion. He was quite sure Mr Campbell would not design such a girder with a square cut.

If a plain simple bearing was used and there was likely to be trouble from corrosion of the bottom flange, the Author suggested that it was not difficult to put a little additional steel—perhaps ½-inch or ⅜-inch plate—

as an extension of the bearing plate in between it and the bottom flange plate, so that any corrosion which took place occurred in that steel.

Turning to the wider question of the general organization of bridge work, the Author strongly agreed with Mr Campbell that the work should not be organized into watertight compartments. It had been rightly said that the D.E. should carry out bridge work, because he was responsible for the safety of the track, and, after all, the work would be done by the district men if it was to be done by the Railway Executive at all.

But watertight compartments could occur. District work on the works side fell into two categories. First, there was the heavy structural work—bridges, tunnels, retaining walls, and other load-carrying structures—and then there was supplementary building work. The distinction which arose was more a distinction between the work of a civil engineer and an architect than anything else. The Western Region organization was such that that cleavage, which was a natural one, was followed. Therefore, one inspector, the Bridge Inspector, dealt with superstructures and substructures, tunnel work, retaining walls, and the provision of the types of timbering which were shoring and protection to the running line.

There were other systems where supervision on the works side was under a Chief Inspector and there was an Inspector of brickwork and an Inspector of steel. In that case one had one inspector dealing with the substructure and the same man also dealing with the more ornamental brickwork of buildings. The steel man would be doing repairs to station roofs and minor semi-decorative work as well as erecting bridge superstructures.

The Author did not think any difficulties in design could arise through lack of knowledge of erection troubles if the D.E. did the erection. He would say rather the reverse.

The other important feature determining one's method of carrying out bridge work and the general scope of the organization was whether the work was to be done by contract or by railway personnel. He had a great predilection for the Railway Executive doing their own work of usual type so far as was humanly possible, because surely the profit which the contractor must make could be just as well in their pockets. Even if they were to go very extensively to the contract system, he did not think it desirable to run bridge work in detail from headquarters, especially on the bigger regions.

Mr Berridge had shown them a good illustration of an Indian crane. The Author very much envied him that crane which could be used to put a 40-foot span ahead of itself and even to drive the piles beforehand to receive it. It was a very useful tool. Perhaps a fundamental difference between Indian bridge work and British was that spans in India were almost all square. Spans had been so standardized that work had probably been reduced to a complete drill. He imagined that in dealing with local labour it was necessary to reduce it to a complete drill. Perhaps that might account a little for the very short occupation times. Certainly, if one could face up to a speed restriction for a very long period of time, then the occupation time could be cut down. The Author was delighted with the illustrations of the well deck. There was no doubt that they were excellent. He had not yet had the pleasure of handling them or seeing them handled.

Mr Smith had stressed that it was simpler to build a new abutment behind the line and the back of the existing abutment rather than through it. The Author agreed. Also Mr Smith had stressed that it was better, if cross-girders were going to be difficult to fix from a crane working below, not to have cross-girders. The Author agreed that breakdown cranes were slow. But if they were to make it a practice only to use them for the bigger lifts, and if for those bigger lifts they were to assume that they were more or less up to capacity, he did not feel any burning desire to increase their speed of lifting. The setting of a big long girder was not a job to rush and if it took 10 minutes longer than if one had a higher gear ratio that was not of great importance. It was only a small part of the operation. If one could have hydraulic rams, and possibly hydraulic jacks, so that the extension girders could be set more quickly, he thought that that would be a very real improvement, in cutting down the time to "dress" the crane, the operation in which their slowness was really revealed.

Mr King had made a point about the direct labour organization. He thought that he could best reply by saying "fair enough": the organization was taken over from the L.M.R. It was a fact that at Bristol they found that the direct labour organization worked extremely well. It had proved a very ready source for increasing their man-power. He agreed that anything that one could do to speed up the passage of traffic over temporary works was highly desirable, but he had not any proved ideas to offer. It seemed to him that they would probably have to go down to a good depth to get a good bearing. They might have to resort to pre-cast slabs or timber going below any deterioration of the original material of the bank. In addition to improved bank seats, the temporary work should permit ballast between it and the sleeper so that a good top could be maintained constantly. Although he agreed that it was an objective at which they should aim he agreed, equally, that he had not got the solution. He was glad that Mr King agreed with him on questions of general organization.

Mr Hawkins had said that their painting at Bristol appeared to be done very cheaply. He did not know whether that was because it was done by direct labour and not by contract. From figures he had seen he knew that at Bristol, by direct labour, the ratio of labour to paint on building painting was 70 per cent labour to 30 per cent paint. By contract the figures were more like 75 per cent labour to 25 per cent paint. That did not sound to be a very great difference but in fact it was considerable.

The other point was that quite a few bridges, especially the older bridges where they still had more or less the Brunel type of girder, were tarred and had been tarred for generations. Obviously, they would be tarred until they were replaced. Tarring, of course, was a cheap operation.

He agreed with Mr Hawkins that asphalt properly applied could be a very good waterproofing system, but it was not particularly cheap. Again, it needed protection and lacked the flexibility of the membrane system. He also heartily agreed with the comments of the Chairman about jacks. There was nothing more important than to have a stand-by, particularly if one was using hydraulic jacks which might give trouble. He recalled a job in South Wales where it had been necessary to lift a five-span bridge, each span carrying four tracks over the Rhondda River. The number of jacks required for the job was about forty-seven. He had acquired all he could by lawful and unlawful means from their own resources and by borrowing. In the end, they got through with about six in hand but he did not feel that that was one too many.

The Author did not consider Mr Wilks's suggestion of the erection at site of special cranes to be in general an economical suggestion because of the variation in site conditions. The availability of breakdown and general-purpose cranes for bridge work was one of the most economical factors in bridge reconstruction work and for that very reason such cranes were almost invariably hired from the railways by contractors, when the erection was let to contract.

The Author noted Mr Wilks's varying taste in the matter of abutment facework and agreed that by proper panelling and the use of oversailing, very pleasing effects could be obtained.

In suggesting pre-stressed concrete decks for temporary work over abutments the Author had in mind the use of cross-sleepered road with a small cushion of ballast, which would allow of much better running and the more consistent maintenance of top, and in that matter the shallow depth possible with pre-stressed concrete work would be a great help.

The Author was aware that students were taught to avoid the use of bolts in tension and, where conditions permitted, a lifting bracket secured to the end plate by bolts in shear was invariably used, but there were many cases where the crane had to pick up the load at an intermediate point on the girder, and the experienced engineer, the Author suggested, was justified then in using bolts in tension and care should be taken by close supervision to ensure that the bolts of the lifting bracket fixed to the top flange were "hand tightened" only.

CORRESPONDENCE
on Papers published in
Proceedings, Part II, February 1952

Paper No. 5833

" Marine and River Surveying with a Circle Chart " †

by

John Francis Causton Swansbourne, A.M.I.C.E.

Correspondence

Commander D. H. Macmillan observed that the Author had rightly emphasized the eminent advantages to be obtained by the use of intersecting arcs and position lines for the location of moving objects or vessels afloat adjacent to shore control points; but a very great deal more, of course, could be said about ensuring optimum conditions of intersection and also the sensitivity of the angle related to linear displacement.

Whilst that principle was almost absurdly simple it did not appear to have been given the place it deserved, despite the fact that Commander Macmillan had developed the system for use in plotting the Liverpool bar in 1932; the subject was dealt with in some detail by him in a publication entitled " Precision Echo Sounding and Surveying " (published by Kelvin & Hughes in 1939).

That, together with some Papers appearing in the Dock and Harbour Authority magazine for April and June, 1939, had elicited a letter from Mr L. D. Shuman, Chief Engineer of the United States War Department, showing that American Surveyors had developed a similar control system for the navigation and surveying of the River Mississippi.

Commander Macmillan was considering a small work on the subject as it could be applied for a variety of purposes quite apart from hydrographic surveying and enabled location to be done on large scales with very small plotting sheets with arcs correctly related to the trigonometrical framework upon which the determinations depended.

A number of valuable relationships existed between the subtended angles and the rate of change of distances and angles of intersection of the arcs and it was also possible to dispense entirely with beam compasses and plot the arcs on very large scales by rigorous calculation.

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† Proc. Instn Civ. Engrs, Part II, vol. 1, p. 210 (Feb. 1952).

The Commander was very interested in the discussion and felt that the utility of the solution had not been fully explored by engineers and surveyors generally, and that a valuable service had been performed by the Author in raising the issue.

The principle had been used for the control of Naval operations under the Commander's supervision during the recent war and it might be of interest that the entire area under the control of the Southampton Harbour Board was covered by sextant arc graphs on a natural scale of 1/2,500 throughout, depending upon the minor triangulation covering the approaches and sea channels leading to the Estuary and Port, as well as the areas off the Docks.

The Author, in reply, stated that he was pleased to note that sextant graphs or circle charts had been used extensively by a naval hydrographer and wished to thank Commander Macmillan for drawing attention to the other treatments of the subject which he had since read with great interest.

All civil engineers who were connected with river, port, or harbour development should read those works. Such extensive experience could only support the contention that civil engineers would do well to be informed in the simple arts of graphical plotting.

The preparation of a chart to suit a given set of shore stations might seem laborious to a civil engineer, particularly if the job was a small one and he was only going to use the chart once. The Author felt, however, that the labour was justified by the result whatever the size of the work, but that some simplification might be found in the preparation of a standard chart, as described in the Paper, for use on any survey where conditions were suitable.

Paper No. 5845

" Construction of a Pre-stressed Concrete Road at Crawley New Town "†

by

Arthur John Whyte McIntosh, B.Sc., M.I.C.E.,
and Jack Mercer, M.Sc.(Eng.), A.M.I.C.E.

Correspondence

Mr W. E. I. Armstrong observed that the pre-stressed concrete road described by the Authors was the first laid in Great Britain and, so far as was known, the longest in the world at that time. It was an outstandingly successful experiment, not only because it had demonstrated that pre-stressed concrete roads could be laid at a cost no higher than that of ordinary reinforced-concrete roads, but also because of the information it gave on movements to be expected on other similar pre-stressed slabs. That information was invaluable to those concerned with the application of pre-stressing to road work.

It should be noted that although the use of separate haunches to the road possibly cost more than the alternatives which might now be used, they served their purpose by keeping down tendon anchorage failures and limiting the severity of failures. Those were important considerations with pre-stressed concrete road slabs. With thin pre-stressed slabs, the maximum compressive stress at an anchorage was normally greater than with thicker slabs, and failure was more likely unless special attention was paid to safety in design and construction. If failure occurred the long tendons in a road slab could release more energy than would be the case in the shorter units commonly used in pre-stressed concrete structures and failure could be more severe. Certain precautions therefore had to be taken whether a haunch was used or not. They included the use of a good factor of safety, special attention to concrete compaction in the anchorage areas, the use of stronger concrete in those areas, and the cutting-out and replacing of badly placed anchorages. In addition, concrete should be prevented from getting on to any anchorage surface on which jacks were subsequently to bear; calculations should be based on the pre-stress being transferred by direct compressive thrust only; fabric reinforcement should be placed at the ends of slabs; and ties should be used at right angles to the tendons near the anchorages. It would be seen how easy it was to comply with all those requirements when a special haunch was used. In addition, the energy stored up in any one tendon was reduced. If a haunch was not used care had to be taken to ensure that tendons were not placed too near the edges of the slabs. Whilst economics and the advantages of running the tendons longitudinally along the road might limit the use of haunches in future, it was doubtful whether sufficient was known about pre-stressed concrete roads at that time to have safely omitted the haunches without other equally or more expensive alternatives.

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† Proc. Instn Civ. Engrs, Part II, vol. 1, p. 220 (Feb. 1952).

When considering safety in design and construction of long thin pre-stressed slabs it was opportune to mention safety during stressing so that those failures which did occur, though of reduced frequency and magnitude, would cause as little damage as possible. The jacks and pump should be operated from well to the rear and the operators should be given some protection against flying wedges, etc. Personnel observing progress or operating the jack should be provided with metal eye shields or basket goggles. The areas forward of any anchorage being stressed and adjacent to the side of the slab near any anchorage being stressed had to be avoided. No one should lean over a jack once stressing was well under way; marks which would coincide when full tension was reached should be made on the tendon and jack; observation from a distance with optical aid might even be considered. Pump pressure-gauges should be frequently calibrated and pulls for different pressure-gauge readings should be known. In addition, boards should be placed over the end of the slab during stressing.

Information was at present lacking as to whether the use of more construction joints would in practice solve the problem of avoiding cracks before stressing when large reductions occurred in the temperature. It was probable that the Authors' other suggestions on p. 232 were better. The use of more construction joints might result in more weak points, since both flexural and shear strength was normally less at those points.

The measurements of slab movement were being continued by the Road Research Laboratory and the Authors were to be thanked for the facilities they had given for that work.

Mr E. H. Cooley observed that the Authors' conclusions were of considerable interest in the light of recent work that had been undertaken by the Cement and Concrete Association on pre-stressed concrete roads. In the course of the past 2 years the Association had laid five pre-stressed slabs, ranging in length from 40 feet to 190 feet, to investigate various aspects of the problem. In those slabs pre-stress had been applied in the longitudinal direction only since it had been thought that a pre-stress in the transverse direction was unnecessary except for very wide slabs.

As concluded at Crawley, it had been found that the most suitable method of producing that longitudinal stress was simply by running a number of cables straight down the length of the slab, anchored positively at each end. The stresses in the corners of the slabs were usually the most critical for normal types of loading and in two slabs an attempt had been made to carry the pre-stress right into the corners by fanning out the wires in the cables from a point 10 feet from the end of the slab. The wires had then been taken into the corner of the slab and anchored by a positive bond with the concrete. That method had worked satisfactorily but had involved rather a large amount of site work. In subsequent slabs the ends had been dealt with by placing a length of mesh fabric over the end 10 feet, the cables being anchored in the normal way.

One problem that arose when long slabs were stressed by longitudinal cables was that friction in the cables had the effect of reducing the stress applied to the concrete. That friction arose from the fact that the heavy vibration that was usually applied to concrete in road construction tended to displace the cable from its desired straightness, or even to damage the sheathing if the cable was laid in situ.

That problem might be overcome by using a fairly heavy gauge sheathing as had been done at Crawley, but that increased the cost of the cables. The method which had been developed by the Cement and Concrete Association and which appeared to work extremely satisfactorily was to form a duct for the pre-stressing cable by a length of galvanized pipe, of diameter slightly greater than the cable, which was pulled through the concrete as work proceeded. The concrete was placed and vibrated at the forward end of the pipe, a sufficient length of which remained inside the concrete to prevent any collapse of the duct until the concrete had taken its initial set. With a 40-foot length of pipe it was possible to lay the concrete in a continuous process, pulling the pipe forward a few feet at a time. The pipe was supported at the right height in the slab by concrete blocks at intervals. The method was suitable for straight cables only, of course, but was being used successfully on a scheme at Basildon New Town in Essex.

From the economic view point it seemed that there was little to choose between a 6-inch pre-stressed concrete and a 6-inch normally reinforced slab. However, where a 6-inch pre-stressed slab, with its greater flexural strength, could be used in place of an R.C. slab of 8 inches or more thickness then some considerable saving was likely to be achieved, in addition to the benefit of obtaining a riding surface with far fewer transverse joints.

Dr P. W. Abeles referred to the statement in the Paper that a " considerable compressive stress should remain in the concrete after making allowance for the reduction of stresses due to sub-grade restraint " (p. 221). It was interesting to note that an effective compressive pre-stress as low as 100 lb. per square inch had been considered as sufficient. That was, in fact, only a small proportion of the flexural strength (also called modulus of rupture or tensile strength in bending) of high-strength concrete as used in pre-stressed road work, which was approximately 800 lb. per square inch, and differed greatly from the usual arrangement in pre-stressed concrete construction where mostly the effective pre-compression was higher than the modulus of rupture.

In a non-pre-stressed reinforced-concrete slab, cracks would occur owing to shrinkage and creep, even if the bending tensile stress under the maximum wheel load was below the flexual strength, but the development of such cracks was avoided by effective pre-stressing, even if the stress was rather low. According to Dr Abeles's suggestion, constructions in which use was made of the flexural strength of the concrete were called partially pre-stressed in contrast to fully pre-stressed structures in which there was only compression in the concrete when under working load.

Tests had proved that partially pre-stressed concrete might be designed so as to ensure freedom from cracks under working load even under fatigue conditions, which case was described as Type (iB) in the "First Report on Prestressed Concrete" (Appendix II),[1] or in such a way that temporary fine cracks might occur under load conditions, which was called Type (ii) in the Report, the cracks closing completely on removal of the load. Thus both types could be successfully used for road constructions and a very small but effective pre-compression would be sufficient to avoid the development of cracks altogether if, under repeated wheel load, the resultant tensile bending stress did not exceed approximately 75 per cent of the modulus of rupture. If, for example, the concrete tensile stress under the maximum wheel load was 700 lb. per square inch in a non-pre-stressed reinforced-concrete slab, it would crack independently of shrinkage and creep; but a pre-stressed slab with an effective pre-compression of 100 lb. per square inch would not crack, the resultant stress being reduced to 600 lb. per square inch, which reduction would just ensure freedom from cracks under fatigue conditions. Or in another case where the concrete tensile stress in a non-pre-stressed slab might be 1,000 lb. per square inch, a low effective pre-compression would ensure that any fine crack which might develop under maximum wheel load would become entirely invisible on removal of the wheel load, even after millions of repetitions of loading. Thus an effective pre-compression as low as 100 lb. per square inch, which was approximately $\frac{1}{80}$ of the cube strength or even less, was sufficient to obtain satisfactory results.

The Authors, in reply, observed that Mr Armstrong had drawn attention to the advantages which resulted from the use of separately constructed haunches. Although it had been stated in the Paper that the haunches could be omitted, they were very conscious of those advantages and the suggestion to eliminate them had been made solely on the grounds of economy. At the same time, the Authors considered that if that were done then some thickening of the edges of the slab would be desirable, because even with the carefully prepared haunches at Crawley two cases of failure of the concrete surrounding the cones had occurred while stressing. Some further experimental work was clearly needed before conclusions could be drawn. Mr Armstrong's remarks on the precautions to be taken during stressing were timely and the Authors felt that a Code of Practice recommending the precautions to be taken during stressing should be produced for the benefit of operators and observers.

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[1] "First Report on Prestressed Concrete." Instn Struct. Engrs, London, 1951.

Mr Cooley's information concerning the longitudinal stressing used by the Cement and Concrete Association was interesting, but it should not be thought that the use of diagonal cables at Crawley had been prompted by the need to apply transverse pre-stress, which was in fact almost negligible. The main purpose of placing the cables diagonally had been for ease of handling with the added advantage of being able to execute all stressing from the sides of the road. The Authors felt strongly that if the full value of pre-stressed road construction were to be obtained in the future, speed of construction would be essential and the method of construction would have to lend itself to the mechanical laying and compaction of the concrete. Methods of stressing longitudinally had not, to the Authors' knowledge, taken any account of that factor, for a bay about 5 to 6 feet long had had to be left for stressing operations and concreted at a later stage. As pointed out on p. 223, the slab had been designed to test the apparent advantages of being able to carry out the stressing at the sides of the road without interruption of the continuous operation of laying concrete. If, however, longitudinal stressing only was to be used, then the Authors felt that consideration should be given to the use of high-tensile bars as a means of applying pre-stress, in which case the problem of sheathing might be eliminated or simplified.

Mr Cooley's statement that the use of a heavy sheathing increased the cost of the cables was considered by the Authors to be doubtful. The cost of the sheathing at Crawley was 3*d*. per foot run (equivalent to 6*d*. per square yard) and it was felt that the total cost of providing and laying that sheathing was probably less than the cost of the process of winching forward galvanized iron piping. Moreover, the latter process could easily lead to a slight disturbance of the green concrete unless very great care was taken. In the last paragraph concerning costs, Mr Cooley was more optimistic than the Authors, who doubted whether the costs of a 6-inch reinforced-concrete slab and a 6-inch pre-stressed slab were comparable. In the Paper, the cost comparison had been made between the normal construction at Crawley of 8-inch reinforced-concrete on 4 inches of lean concrete as against a 6-inch pre-stressed slab on clinker. Those costs were felt to be comparable and with greater experience might show an advantage in favour of the pre-stressed road.

The contribution by Dr Abeles was considered by the Authors to be of great value, affording as it did a theoretical basis for the low value of the pre-stress used. As pointed out in the Paper, however, it was felt that a higher pre-stress of (say) 300 lb. per square inch should be used until more information was available concerning the value of the subgrade restraint on long lengths of continuous concrete road.

CORRESPONDENCE
on Papers published in Proceedings, Part II, June 1952

Road Paper No. 36

" Some Theoretical Aspects of Road Traffic Research " †

by

John Glen Wardrop, B.A.

Correspondence

Mr J. I. Whitehead agreed with the Author that, although traffic engineering was a severely practical subject, some theory was required to guide the interpretation of observations. In Great Britain, the actual decisions on action to be taken in relation to traffic flow were made by laymen—the councillors of the local authorities who had highway powers. The appearances and properties of traffic could be extremely deceptive to the casual observer, and far too many decisions had been made in the past without ascertaining the true facts in particular traffic problems. It was therefore very important that the council's technical advisers should have an opportunity, such as was given by the Paper, of studying some of the basic facts of traffic flow.

In studying speeds, the Author had emphasized the need for keeping to the same type of mean speed in any investigation. It appeared to Mr Whitehead, however, that the facilities for observing space-mean speed were very limited, and that time-mean speed was almost invariably used because its measurement was simple. The risk of confusion was not therefore likely to be great, although the Author's warning was justified.

The Author had mentioned the effect of the high correlation between the journey times of successive vehicles passing along the same route within short time intervals. Mr Whitehead had to make observations of journey times to obtain time-mean speeds to assess the need for imposing 30-mile-per-hour speed limits. In doing so, the only vehicles observed had been those whose speeds did not appear to be influenced by the next vehicle ahead. That method probably gave higher average speeds than if all vehicles were included, and he would like to have the Author's opinion as to the merits of this method. It had been used because the public always attached great importance to isolated vehicles travelling at high speeds, and it seemed necessary to measure those because they were the basis of the complaints which led to demands for speed restrictions.

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† Proc. Instn Civ. Engrs, Part II, vol 1, p. 325 (June 1952).

The need for a knowledge of elementary statistics for traffic research work was rightly emphasized by the Author, and Mr Whitehead thought that that knowledge might well be regarded as essential to all those engaged in traffic engineering. It was clearly impossible to give many worked examples in a Paper such as the one under discussion, but they would have been welcomed because most text-books related the subject to biology, and it was not easy for those unskilled in the subject to select the right method for studying the observations in any particular traffic problem.

The Author, in reply, referred to Mr Whitehead's suggestion that in traffic studies the time-mean speed was almost invariably used because its measurement was simple. Experience at the Road Research Laboratory, however, showed that the most common methods of speed measurement involved timing vehicles over a fixed distance (which might be anything from 3 feet to a mile). The results were measured directly in terms of journey time, and in order to obtain the space-mean speed it was only necessary to find the average of those journey times and divide that into the distance.

The time-mean speed, on the other hand, required each individual journey time to be divided into the distance and the extra work would be considerable if large numbers of vehicles were involved. An additional advantage of the space-mean speed was the comparative ease with which it could be calculated mechanically using punched-card equipment.

The Author agreed with Mr Whitehead that the mean speed of those vehicles which appeared to be unaffected by other traffic would generally be higher than that of all vehicles. Measurements restricted to those vehicles should therefore only be used when attention was directed to the faster traffic, as in the example given by Mr Whitehead.

Airport Paper No. 19

"The Influence of Multiple-wheel Undercarriages on the Design and Evaluation of Airfield Pavements"†

by

Guthrie Stewart Cooper, B.Sc., A.M.I.C.E.

Correspondence

Mr G. M. J. Williams observed that the Author appeared to have assumed that the criterion of failure in the soil below a flexible pavement was the maximum total normal stress acting in a vertical direction. Most authorities, however, considered the criterion of failure to be whether or not the shear stress on any plane at any point in the soil exceeded the value of the expression $c + \sigma \tan \phi$, where c denoted the cohesion and ϕ the angle of internal friction of the soil, and σ denoted the direct stress on the particular plane. Thus to derive the combined effect of several surface loads one should compound the shear stresses produced by each load and also compound the direct stresses on the planes of maximum shear stress.

The Author also appeared to have assumed that failure of the subgrade was dependent only upon the stresses produced by the combined loading and upon the classification of the soil (that was, its cohesion, angle of internal friction, and other physical properties), but that it was independent of the depth at which failure occurred. Thus if a single-wheel load set up a maximum stress in the soil which just caused it to fail, then the combined effect of two lesser wheel loads would cause the same soil to fail if they produced the same maximum stress at any point, even though that point might be deeper than the point where the maximum stress occurred under a single-wheel load. In soils possessing internal friction that would not be true because, as was well known, the apparent strength of such soils increased with the depth even though the angle of internal friction did not change. In such a case the safe load on a multiple undercarriage should be greater than that computed by the Author. On the other hand, on cohesive soils the pre-existing stresses in the soil might cause a reduction of the apparent strength with depth below a compacted subgrade. Both of those factors had been dealt with in detail in recent literature on the subject.[1]

Finally, Mr Williams considered that it would be of interest if the Author would give his reasons for assuming that the criterion for failure of a rigid pavement was usually the flexural stress in the bottom of the pavement. It was true that failure always became apparent by failure of the pavement but there was no sure method of telling whether or not failure first occurred in the subgrade and was followed by failure of the pavement itself. When designing flexible pavements by the Westergaard method, or any other method which assumed that failure would first occur in the pavement, it was desirable to check that the stresses set up in the subgrade were also safe. That was particularly necessary in cases where the pavement would undergo a considerable deflexion before failure, such as arose with pre-stressed-concrete pavements, or when the total loaded area was large compared with the pavement thickness, as tended to happen with multiple-wheel undercarriages.

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† Proc. Instn Civ. Engrs, Part II, vol. 1, p. 419 (June 1952).

[1] G. Wilson and G. Williams, "Pavement Bearing Capacity Computed by Theory of Layered Systems." Trans. Amer. Soc. Civ. Engrs, vol. 116 (1951), p. 750.

Mr R. S. Colquhoun observed that the Author had stated on p. 440, that temporary airfields surfaced with metal tracking, such as "Pierced Steel Plank," presented a difficult problem. Would the Author please state whether such metal track-surfacings should be regarded as "rigid" or as "flexible" pavements? Had he any experimental data which would show the extent to which such surfaces spread the wheel load over the natural surface of the ground? It was understood that a runway of Pierced Steel Plank in combination with Prefabricated Bituminous Surfacing was in constant operational use at Changi Airfield, Singapore, from 1945 to 1949. Could the Author state what were the subgrade and moisture conditions of that runway, the type and weight of aircraft using it, and the number of coverages during the period of operation? Had he any information on the effect of multiple-wheel undercarriages on Channel Track surfacing (where the individual panels were 6 feet by 11 feet) where the contact areas of the two wheels would be confined within one panel?

Such information would be very useful in evaluating the performance of metal track-surfacings, on which subject there was little published data.

Mr R. L. Mitchell, of Salisbury, Southern Rhodesia, considered that although the Author was to be congratulated on having presented a very useful thesis on the effects of multiple-wheel undercarriages, the adoption of that thesis to rigid pavements could only be regarded as academic. The Author had stated, on p. 429, that "slab corners are, accordingly, the weakest parts of the structure . . . [and are] taken for the calculation of thickness . . .", with which Mr Mitchell agreed, yet the Author's subsequent calculation were concerned with the more readily soluble centre case. In the discussion[1] on a previous Airport Paper, Mr Mitchell had called attention to the possibility of modifying Westergaard's formulae for corner loading to cover multi-wheel undercarriages. The Author had reported the limitations of that case, when calculated from a heavy equivalent single wheel load, and had reverted to the centre case; that was equivalent to stating that if a multi-wheel undercarriage were replaced by a single wheel, then the formulae would not be valid for corner design, and that only the centre design should be considered—or in other words, corner design should be used for small aircraft, centre design for large!

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[1] J. K. Fisher, A. Goode, and C. E. Docker, "Some Problems in the Design and Construction of Large Airfields." Airport Paper No. 3, Instn Civ. Engrs, 1946. *Discussion* by R. L. Mitchell, p. 74.

Mr Mitchell felt that the approach was fundamentally incorrect, particularly when considering the E.S.W.L. on rigid pavements. The worst loading would occur when the front wheel(s) was on the slab and the rear wheel(s) just on the corner, and he estimated that it would occur near the edge with the axis of the wheels parallel to the sides of the slab. Could not each wheel be considered separately, and a total summation of stress be evolved? A more practical consideration would, however, evolve in questioning what brought about corner failure; in Mr Mitchell's opinion it was caused by the impact of aircraft wheels on proud corners. In that aspect the multi-wheel undercarriage would have the advantage, because it would dampen the impact stresses. Also, since moisture and temperature conditions so much affected the stresses, and since so little was known with regard to the stressing of load-transfer devices, he considered that rigid-pavement design was best carried out empirically, as with the C.A.A. (Civil Aviation Authority, U.S.A.) system, by which a nominal reduction would be made for multi-wheel undercarriages. Research should enable a world-wide standard to be laid down for mix design; with that as a standard, records could be kept of subgrade classification (or, preferably, of moduli of subgrade reaction), pavement thickness, climate, and aircraft causing failure, and an empirical classification could be established.

Mr Mitchell thought that, although rational, the E.S.W.L. was too complicated a function to be of primary use. From an operational point of view, it would be better if all manufacturers of aircraft with multi-wheel undercarriages were to issue to all governments the C.B.R. and depth/vertical-stress curves; those would then be in the hands of authorities before operation of the aircraft could be anticipated, and would give them sufficient time to classify their airfields for use of the particular aircraft.

The Author, in reply, agreed that the shear stress was the ultimate criterion for failure in a flexible pavement. In establishing the Equivalent Single Wheel Load Ratio for a particular pavement, however, c and ϕ were constants. The stressing conditions in the subgrade were considered since it had been assumed at the outset that that was the critical plane for failure, also that the single and multiple wheel loads which produced the same average vertical pressure over a defined area would produce roughly the same shear stresses. At an early stage in the investigations efforts had been made to equate shear stresses, but it had not been found practicable by that means to produce a method which would be sufficiently simple for general use.

It was true that failure might occur elsewhere than in the subgrade under various wheel arrangements, and that point had been mentioned i the last paragraph of p. 423 and the first paragraph of p. 440 in the Pape and also at the commencement of the Author's reply to the Discussion

The Author stated that, whilst excessive flexural stress in the bottom of a concrete slab was the cause of slab cracking in the case of centr loading, cracking in concrete slabs usually began at the corners and edge owing to excessive flexural stress in the top of the slab. The reason fo adopting the centre case to establish the Equivalent Single Wheel Loa Ratio was given on p. 430 in the Paper. The Author considered tha since cracking of concrete pavements usually occurred at deflexions of les than 0·15 inch, it was very unlikely that the safe load on the subgrad would be exceeded. Pre-stressed concrete pavements introduced completely different problems and the method given in the Paper for th calculation of Equivalent Single Wheel Load Ratios for rigid pavement could not be applied directly. Pre-stressed concrete, permitting muc higher deflexions, allowed greater benefit to be derived from the natura bearing strength of soil. The pavements as a whole combined th characteristics of rigid and flexible pavements, and a method of calculating Equivalent Single Wheel Load Ratios for them would have t be evolved.

Regarding landing mats, the Author stated that fundamental researc on the whole field of prefabricated surfacing was at present in hand, bu he was not in a position to discuss it. Various types of those surfacing were under development and, according to type, the pavement as a whol ranged from purely flexible to partially rigid. Channel Track was no now in use. At Changi two layers of Pierced Steel Plank were used, th lower layer being placed longitudinally and upside-down on top of tw layers of Prefabricated Bituminous Surfacing. Other details of the pavement were available in Mr Goode's Paper.[1] It performed fairly satisfactorily for the period over which it was in use.

The Author agreed that the method put forward for Equivalent Singl Wheel Load Ratio determination on rigid pavements was academic. H could not, however, suggest any other method, and the proposals pu forward by Mr Mitchell did in fact call for individual investigation from first principles in each case. The Author's intention had been to presen a method which would be sufficiently accurate for general and simple use.

The Author stated that, eventually, empirical design methods would undoubtedly be adopted for rigid pavements, but considerably more practical evidence was required before that could be done. The C.A.A system of design was a step in that direction, but the Author thought tha the base course requirements were exaggerated, bearing in mind the very limited deflexion permissible in a concrete pavement.

The suggestion regarding the provision by aircraft manufacturing firms of C.B.R. and depth/vertical pressure curves would only provide one-half of the answer, and the Author doubted that the companies would be in a position to construct the curves. However, they could provide the necessary data on wheel loads, tire pressure, and lay-out, which would permit the civil engineer to evaluate the effect on various airfield pavements as required.

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[1] Alfred Goode, "The Construction of Changi Airfield, Singapore." Airport Paper No. 14, Instn Civ. Engrs, 1951.

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